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APPLICATIONS FOR ADMISSION AND TRANSFER

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		Discussion closes
Forests and Stream Flow. *W. G. Hoyt and H. C. Troxell*	Aug., 1932	
Discussion	Sept., Nov., Dec., 1932, Feb., Mar., Apr., May, 1933	Closed
The Martinez-Benicia Bridge. *W. H. Kirkbride*	Sept., 1932	
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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ON THE BEHAVIOR OF SIPHONS

By J. C. STEVENS,[1] M. Am. Soc. C. E.

Synopsis

Results of field tests of three siphons having nominal capacities of 100, 250, and 500 sec-ft, respectively, are presented in this paper. Important effects of cavitation are disclosed. Pressure gradients as well as pseudo, and true, energy lines are given. A simplified conception of the action of siphons is presented, involving an analogy to free flow and submerged orifices. There is also offered a method of comparing siphons by a standardized coefficient of flow and a standard expression for the efficiency of a siphon.

The Leaburg Siphons

The Leaburg Hydro-Electric Plant, of the municipal power system of the City of Eugene, Ore., consists essentially of a dam in the McKenzie River that raises the water 20 ft for diversion into a power canal, 5 miles long, with a capacity of 2 200 sec-ft. The canal ends in a forebay from which water under a 90-ft head is passed through 12-ft concrete penstocks and 10 000 hp water-wheels, and returned to the river through an excavated tail-race.

The wasteway from the forebay consists of seven siphons, having an aggregate capacity of 3 000 sec-ft, sufficient to relieve the canal should a sudden stoppage of the turbines occur under full flow and during a period when the areas above the canal are draining heavily into it.

Since the plant has only one generating unit installed at present, three of the channels leading from the siphons have not been completed although they are available for use in an emergency.

Fig. 1 shows the inlet to the siphons. In Fig. 2, the water seen in the outlet channel of Siphon No. 1, is from the sluice-gate shown in Fig. 1.

Note.—The subject of "Losses in Siphon Spillways" is one of those selected by the Special Committee on Irrigation Hydraulics for study and research. This paper was presented to the Committee by its author. The Committee has recommended its publication by the Society in order to elicit discussion, in the hope that the results of other field tests of siphons may be made available to the profession. Discussion on this paper will be closed with **November, 1933**, *Proceedings*.

[1] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

The trash from the racks may be sluiced to the river in the channel entering the outlet from Siphon No. 7 at the left.

Three sizes of siphons were used in order to obtain elasticity of performance. Siphon No. 7 (Fig. 2), was designed for a nominal capacity of 100 sec-ft; Siphon No. 6, for 250 sec-ft; and the other five, for 500 sec-ft each.

Fig. 3 and Table 1 show the essential dimensions of the siphons measured after they were completed. The primer consists of a steel gate held between guides, which can be set with its lower edge at or above the crest of the siphons.

The partition walls at the immediate outlet channels of the siphons (Fig. 2), were left unfinished until after tests were made, in order that the siphon barrels could be extended, or the outlets nozzled, or any other alterations made that the tests might indicate as desirable to improve the conditions of flow.

In order to determine the performance of each size of siphon, a set of four piezometers was installed in each of six sections in Siphons Nos. 5, 6, and 7. The sections chosen were relatively the same for each siphon, as shown in Figs. 3(*a*), 3(*b*), and 3(*c*). At each section the piezometers are disposed as shown in Fig. 3. Pipes were brought out to the face of the concrete and connected to a header with stop-cocks so that the pressure in each piezometer could be obtained on a single manometer.

On the inside face of the siphon barrel the pipe was laid flush with the forms. After the forms were removed, the face was rubbed down until the metal and concrete were smooth and in a uniform plane at right angles to the direction of flow.

The crests of the siphons are all at the same elevation, 734.0, which is the normal operating level of the forebay. The priming gate on any siphon may be raised to put that one out of service. When so raised, water flows over the crest as if it were an open spillway. By adjusting the elevation of the lower edge of the primer gate, the sequence of starting and stopping of siphons can be controlled.

There is a recess on three sides of the barrel above Section 3 in each siphon, the purpose of which is to admit air under the nappe when a thin sheet of water is flowing over the crest. This equalizes the pressure within the siphon on both sides of the nappe and permits the water to leave the concrete and fall across the barrel, accelerating the removal of air.

Behavoir in Operation

The plant and siphons had been in operation more than two years before the tests were made. In operation the siphons functioned better than was anticipated. Experience has shown that if the priming gates of all siphons are at their lowest point (Elevation 734.0), the siphons will prime in the order of their size when water rises in the forebay; that is, Siphon No. 7 primes first with a nominal head of 0.2 to 0.3 ft over the crest; Siphon No. 6 primes next with a head of 0.3 to 0.4 ft. The order of priming of the large siphons is somewhat erratic if all priming gates are down. For days, Siphon No. 5 will be the next in order, starting with a head of 0.4 ft when,

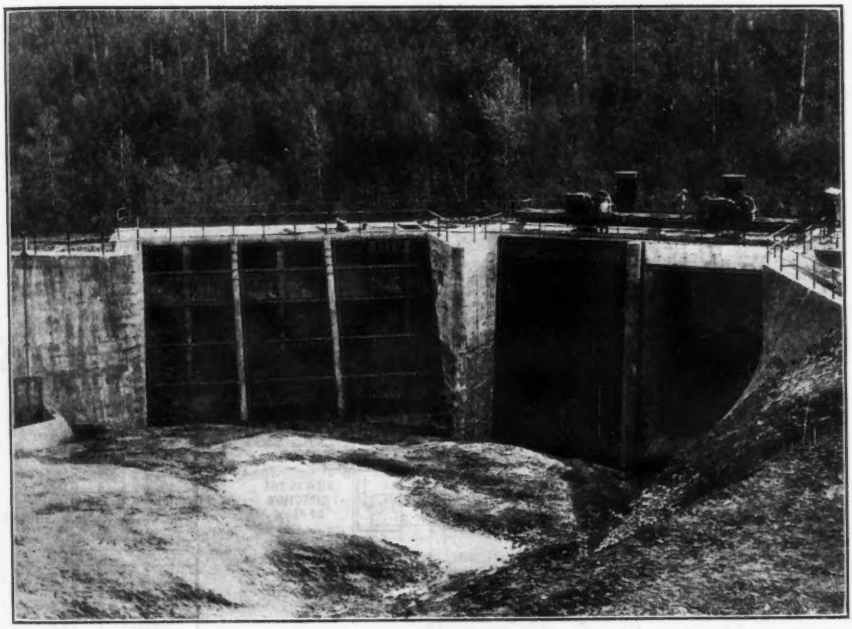

FIG. 1.—FOREBAY STRUCTURE; AT LEFT IS INLET TO SEVEN SIPHONS; AT RIGHT, INLET TO PENSTOCKS.

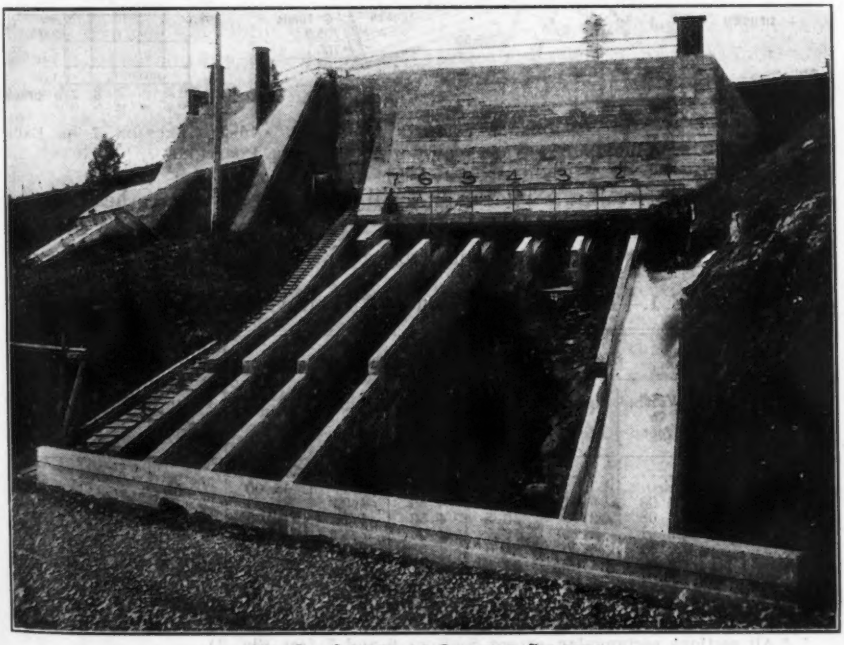

FIG. 2.—OUTLET OF LEABURG SIPHONS.

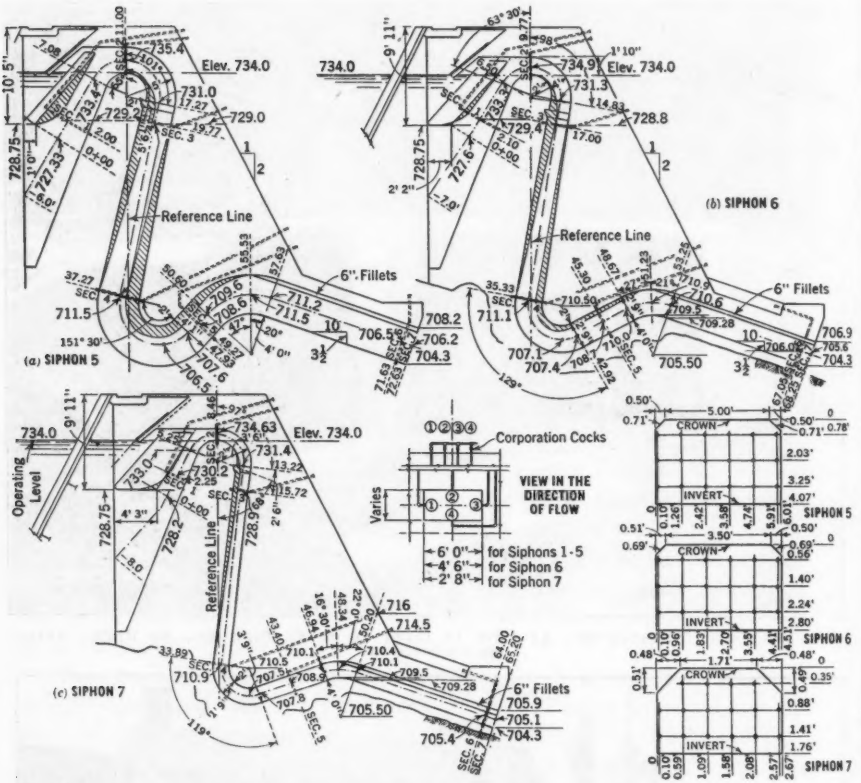

FIG. 3.—ESSENTIAL DIMENSIONS OF SIPHONS NOS. 5, 6, AND 7. SECTION 7 OF EACH SIPHON ENLARGED TO SHOW PITOT TUBE POSITIONS.

with no apparent reason, Siphon No. 1 will start under the same head instead of Siphon No. 5. The priming gates on Siphons Nos. 2, 3, and 4 are kept raised so that they would only come into action as an emergency spillway.

TABLE 1.—ESSENTIAL DIMENSIONS,* SIPHONS NOS. 5, 6, AND 7.

Section †	Siphon No. 5			Siphon No. 6			Siphon No. 7		
	Width, in feet	Height, in feet	Area, in square feet	Width, in feet	Height, in feet	Area, in square feet	Width, in feet	Height, in feet	Area, in square feet
1 primer	.....	.....	9.0	.....	.....	6.75	.....	.....	3.3
1 barrel	.....	.....	30.8	.....	.....	18.00	.....	.....	5.7
Total	6.03	5.01	39.8	4.50	4.01	24.75	2.66	2.14	9.0
2	6.03	2.75	16.5	4.52	1.77	8.00	2.68	1.26	3.43
3	6.01	3.18	19.1	4.53	2.12	9.60	2.69	1.58	4.20
4	6.02	4.09	24.6	4.53	2.78	12.50	2.69	1.75	4.70
5	6.00	4.02	24.1	4.52	2.78	12.55	2.69	1.77	4.77
6	6.01	4.07	24.2	4.55	2.80	12.07	2.67	1.76	4.46
7	6.01	4.07	24.2	4.51	2.80	12.42	2.67	1.76	4.43

* All sections rectangular, except Sections 6 and 7 (see Fig. 3).

During the operation of the plant the quantity diverted at the dam is so regulated that Siphon No. 7 runs almost continuously and Siphon No. 6 operates at times of rejecting load only. The largest siphon seldom comes into action.

A most gratifying behavior of the siphons is the facility with which the flow through them automatically adjusts itself to the surplus inflow to the forebay. As the level of the water in the forebay drops below the edge of the priming gate, air is sucked into the barrel in rhythmic gulps at intervals of about 1 sec. This mixture of air reduces the flow, which arrests the drop of the water in the forebay. Once it has been started, the flow through any siphon may thus diminish to a small part of its normal capacity without breaking its siphonic action.

The result is that almost regardless of the flow into the forebay, or of that taken by the turbine, the water level in the forebay rarely varies more than 0.10 ft above or below its normal operating level of 734.0. These siphons are true "auto-leveling siphons," as they have been called in Italy.[2]

One of the chief concerns in the tests was to be sure the siphon under test was not sucking air. This condition could be met if the water level in the forebay was kept high enough to insure the lower edge of the priming gate being substantially submerged.

Methods of Testing

Tests of the performance of the siphons were made from November 19 to 22, 1931. The flow was measured by a Pitot tube at the outlet of the siphon barrel. Guides were built over the outlet so that the nozzle of the Pitot tube could be held in any position in the verticals shown in Section 7 of each siphon (Fig. 3). These verticals are spaced 0.10 ft from each side and then one-fifth of the remaining width.

In general, velocities were observed at two-tenths, five-tenths, and eight-tenths, of the depth of the barrel. On Siphons Nos. 6 and 7, those at two-tenths the depth at each side vertical were omitted on account of the fillets.

The Pitot nozzle proper was 3 in. long with a $\frac{1}{8}$-in. orifice. It was held in the plane of the outlet of the barrel and parallel to the direction of flow by a length of $1\frac{1}{2}$-in. extra strong pipe which terminated in a connection for, and was joined to, a length of small hose. The other end of the hose terminated in a short glass tube. This glass tube was held beside a level rod and the water level in the tube was recorded at each position of the Pitot nozzle. The elevation of the water in the tube above the water surface at the outlet of the siphon barrel is the velocity head of the water velocity at the position of the nozzle of the Pitot tube. A coefficient of 98% was used for this nozzle. To avoid errors from accumulated air, the hose line was "bled" constantly, except at the instant of reading.

[2] "Experimental Research on Auto-Leveling Siphons," Second Report, by Prof. Guilio de Marchi and Ing. Mario Marchetti, *L'Energia Elettrica*, November 1931; *Technical Memorandum No. 285*, translation from the Italian by the Office of Education, Washington, D. C.

The flow in Siphon No. 7 was first measured by observing velocity heads on a mercury manometer, and, later, by direct observations, in feet of water, as described. The flows obtained were 119 sec-ft to the nearest unit in both cases, although the individual velocities varied somewhat (see Fig. 4(*a*)).

The vertical velocity curves are of considerable interest. Fig. 4 shows them for each siphon outlet. The mean velocity in each vertical was determined by planimeter, and these means were used to form a horizontal velocity

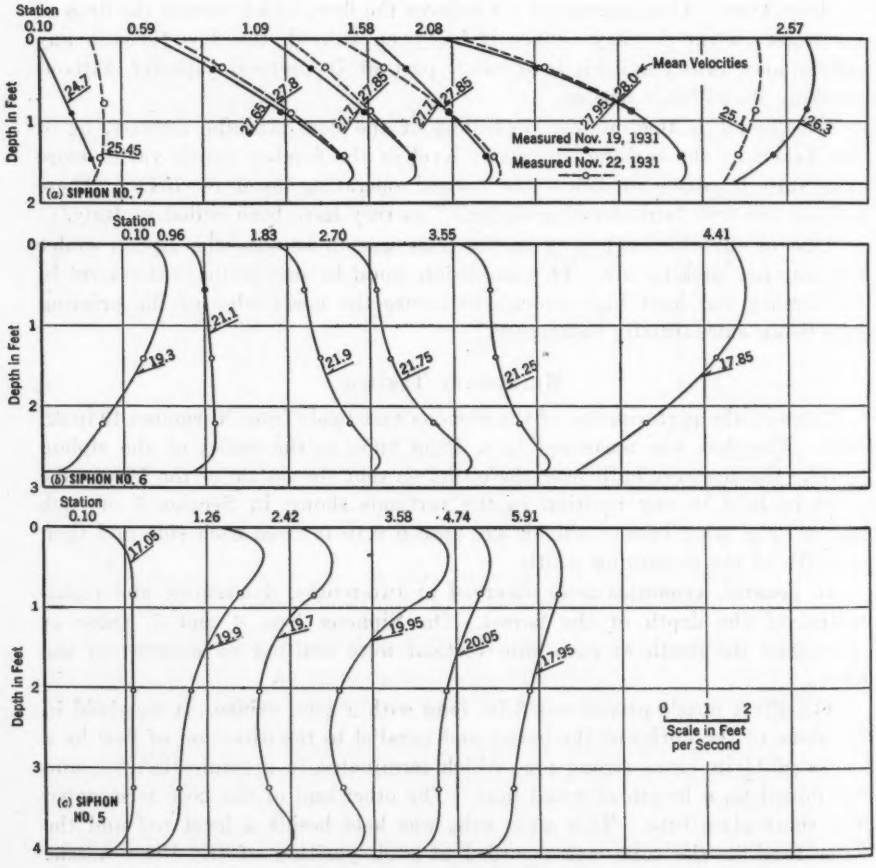

FIG. 4.—VERTICAL VELOCITY CURVES.

curve (see Fig. 5). The arrows and figures show the position and velocity of the mean thread in each vertical. The variations in velocities from the two measurements on Siphon No. 7 are seen to be compensating. The mass of flowing water oscillated rhythmically slightly from side to side, due perhaps to conditions at the inlet.

The variation in the shape of the vertical velocity curves between siphons shows strikingly the effect of the curves in the siphons. The mass swings from one face to the other as it passes through the siphon. The position of the mass of maximum velocity is dependent entirely upon the flow and the proximity of the plane of measurement to the curve up stream from it.

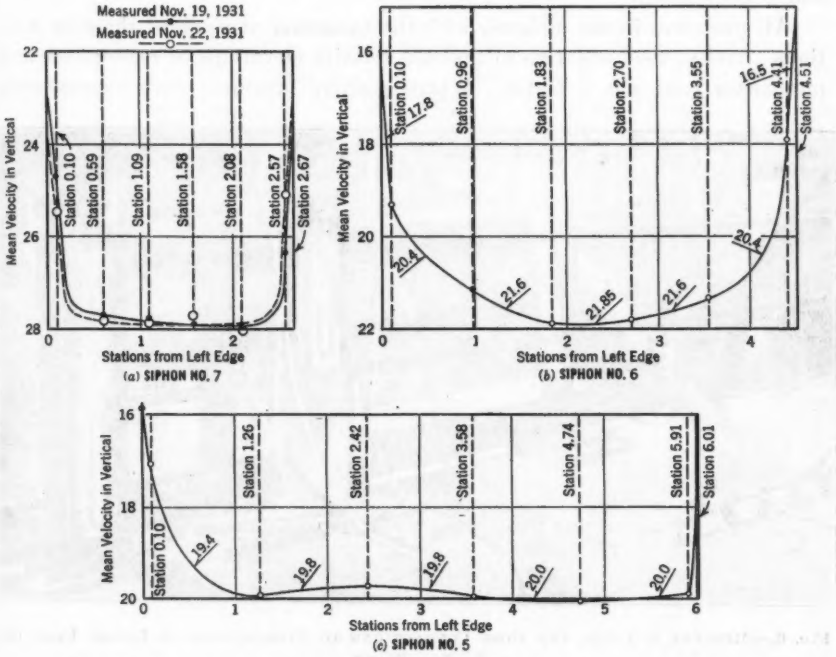

FIG. 5.—HORIZONTAL DISTRIBUTION OF MEAN VELOCITIES IN THE VERTICAL.

During tests, the water issued from the barrel in a solid prism of clear water very similar to a fire stream at the nozzle end. Nevertheless, it is practically certain that Siphon No. 5 was carrying considerable entrained air. The measured flow was 470 sec-ft, whereas it is believed to be capable of discharging 500 sec-ft, or more. The I-beam supporting the rack caused a drop in the water surface just before it enters the primer. This was more noticeable with the large siphons than with either of the others. Air was seen to be sucked in small whirlpools during the test. The outlet, however, showed no evidence of air.

Pressures were read on a mercury U-tube, or manometer. One manometer was fastened to the hand-rail at the top of the siphon block and another to the rail at the deck over the outlets. Each manometer commanded three sections. Pipe fittings terminated in a $\frac{3}{8}$-in. gauge cock. The hose used was $\frac{1}{2}$-in. round rubber packing, having a $\frac{1}{8}$-in. hole which admirably fits the nipple end of this particular gauge-cock. This packing makes an excellent

hose for both positive and negative pressures. It will stand a pressure of more than 100 lb without blowing the packing off the nipple, so that clamps or wiring are rarely required at joints. Fig. 6 shows two sizes of manometers used in the tests. By turning the proper stop-cocks, the pressure in any one piezometer or in the header at each of three sections could be read from a single manometer.

All pressures in the siphons, with the exception of some of those in Sections 1 and 6, were negative so that the relative elevations of manometer and piezometer were not a factor. Where positive pressures were encountered,

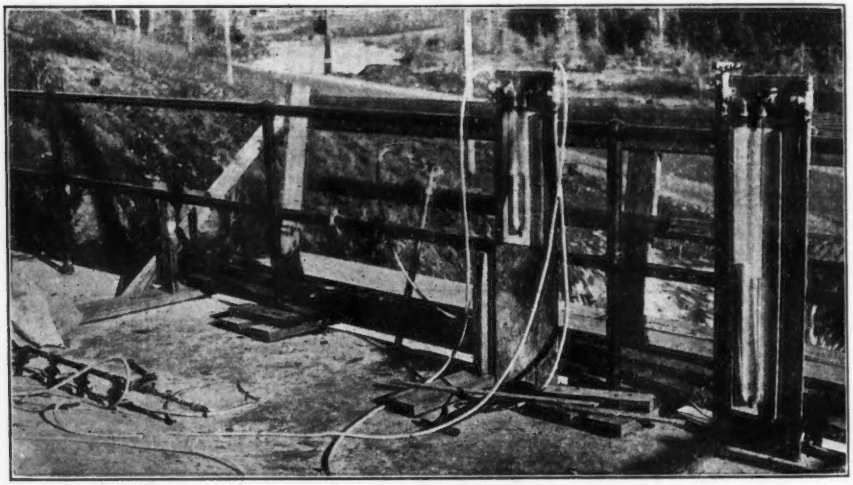

FIG. 6.—MERCURY U-TUBES AND HOSE CONNECTIONS TO PIEZOMETERS ON LOWER DECK OF SIPHON BLOCK.

air was forced into the rubber hose with a tire pump, expelling the water in the piezometer pipe and giving the true pressure on the mercury column.

Oscillations in the water and mercury columns were troublesome. They were dampened by inserting a by-pass fitting in the hose line which was closed by a plug through which a $\frac{1}{64}$-in. hole was drilled. Cocks were included so that the open leg could be used for testing and "bleeding" and the by-pass for reading pressures. This device proved to be very effective and insured a "live" hose line at all times.

RESULTS OF THE TEST

Table 2 gives for each siphon the pressures obtained from each piezometer. Those marked "Header" are the pressures from all piezometers in one section when all are open to the header. The elevation of each piezometer orifice (where the pipe meets the barrel of the siphon) is given, with the elevations of the pressure lines.

Table 3 is a summary of the essential data obtained by the tests. The pressures in Section 6 are omitted on account of uncertainties due to oscillations and to near-zero pressures. The portion of the flow that passed through the primer has been estimated. Elevations of the pressure line and of the energy lines, *A* and *B*, are in absolute values; that is, an atmospheric head of 33 ft has been included.

TABLE 2.—PRESSURE READINGS OBTAINED FROM PIEZOMETERS AT EACH SIPHON

Section	Piezometer No.	Siphon No. 7			Siphon No. 6			Siphon No. 5		
		Elevation of piezometer orifice	Pressure head	Elevation of pressure line	Elevation of piezometer orifice	Pressure head	Elevation of pressure line	Elevation of piezometer orifice	Pressure head	Elevation of pressure line
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Forebay	.....	.....	.....	767.3	.....	.....	767.3	.....	.....	767.6
1	1	730.2	—2.8	760.4	729.4	0	762.4	729.2	—1.5	760.7
	2	730.7	—2.9	760.8	730.4	—0.2	763.2	730.9	—3.8	760.1
	3	730.2	—2.8	760.4	729.4	—0.2	762.2	729.2	—1.1	761.1
	4	729.7	—2.8	759.9	728.4	—0.2	761.2	727.5	—0.2	760.3
	Header.	730.2	—3.0	760.2	729.4	—0.2	762.2	729.2	—1.6	760.6
2	1	734.6	—23.8	743.8	734.9	—13.8	754.1	735.4	—14.5	753.9
	2	735.2	—17.0	751.2	735.8	—9.6	759.2	736.8	—10.9	758.8
	3	734.6	—25.4	742.2	734.9	—14.6	753.3	735.4	—14.1	754.3
	4	734.0	—30.9	736.1	734.0	—24.3	742.7	734.0	—28.8	738.2
	Header.	734.6	—25.6	742.0	734.9	—15.4	752.5	735.4	—16.2	752.2
3	1	728.9	—28.5	733.4	728.9	—32.0	729.9	729.0	—28.7	733.3
	2	728.8	—28.5	733.3	728.6	—32.5	729.1	728.5	—28.0	733.5
	3	728.9	—31.0	730.9	728.9	—32.4	729.5	729.0	—31.5	730.5
	4	729.0	—30.3	731.7	729.2	—32.4	729.8	729.5	—30.7	731.8
	Header.	728.9	—31.1	730.8	728.9	—32 4	729.5	729.0	—30.4	731.6
4	1	710.9	0	743.9	711.1	—2.1	742.0	711.5	—1.4	743.1
	2	710.8	—2.1	741.7	710.8	—2.2	741.6	710.9	—7.5	736.4
	3	710.9	+0.3	744.2	711.1	—2.2	741.9	711.5	—1.1	743.4
	4	711.0	+0.8	744.8	711.4	—2.2	742.2	712.1	0	745.1
	Header.	710.9	—1.2	742.7	711.1	—2.2	741.9	711.5	—5.6	738.9
5	1	708.9	—0.7	741.2	708.7	—1.6	740.1	708.6	—1.8	739.8
	2	709.7	—0.4	742.3	710.0	—2.9	740.1	710.1	—3.3	739.8
	3	708.9	—0.3	741.6	708.7	—2.6	739.1	708.6	—3.3	738.3
	4	708.1	—0.1	741.0	707.4	—2.6	737.8	707.1	—1.1	739.0
	Header.	708.9	—0.6	741.3	708.7	—2.8	738.9	708.6	—3.7	737.9
6	1	705.4	+0.1	738.5	706.0	—0.9	738.1	706.5	+0.2	739.7
	2	706.2	+0.1	739.3	707.3	—1.2	739.1	708.4	—0.7	740.7
	3	705.4	+0.1	738.1	706.0	—1.1	737.9	706.5	+0.4	739.9
	4	704.6	+0.1	737.7	704.7	—1.1	736.6	704.6	+1.6	739.2
	Header.	705.4	+0.1	738.5	706.0	—1.1	737.9	706.5	+0.9	740.4
Outlet	.....	.....	.....	738.9	.....	.....	739.9	.....	.....	741.2

Piezometers Nos. 1 and 3 are on the center lines of the siphon barrels at the sides, while Piezometers Nos. 2 and 4 are at the crowns and inverts, respectively (Fig. 3(*d*)). Theoretically, the average of the pressure-line elevations from Piezometers Nos. 1 and 3 should be practically the same as the averages of Piezometers Nos. 2 and 4; also, the average of Piezometers Nos. 1, 2, 3, and 4 should be nearly the same as when all are open to the same header, as indicated under "Header" in Tables 2 and 3. An examination of the elevations in Table 2 will show that this condition is approximately fulfilled. Such departures of the "header" readings from the average of all piezometers as exist may be readily attributed to oscillations.

DISCUSSION OF RESULTS

If the flow be divided by the area of each section (Column (3), Table 3) and the resulting velocity head be added to the pressure line (Column (6),

Table 3), the elevation of a point on Line *A* (Figs. 7, 8, and 9) is obtained. Line *A* may be thought of as a false energy line, because it is seen to increase above some preceding value. A true energy line must drop continually in the direction of flow.

TABLE 3.—SUMMARY OF ESSENTIAL DATA

Section	Station	Area	Velocity, V_a	Velocity head, h_a	Elevation of pressure line	Elevation of Line *A*	Elevation of Energy Line *B*	Velocity head, h_b	Velocity, V_b	Useful area, *a*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
SIPHON NO. 7: FLOW, 119 SECOND-FEET (19 CUBIC FEET PER SECOND THROUGH PRIMER)										
Forebay	.....	.....	.....	.....	767.3	767.3	767.3	0	.....	.....
1	2.25	9.00	13.2	2.7	760.2	762.9	766.4	6.2	20.0	5.0
2	8.46	3.43	34.6	18.6	742.0	760.6	760.6	18.6	34.6	3.43
3	15.72	4.23	28.1	12.3	730.8	743.1	758.7	28.2	42.6	2.80
4	33.89	4.70	25.2	9.9	742.7	752.6	751.8	9.9	25.2	4.70
5	43.40	4.77	25.0	9.7	741.3	751.0	751.0	9.7	25.0	4.77
Outlet	65.20	4.43	26.9	11.2	738.9	750.1	750.1	11.2	26.9	4.43
SIPHON NO. 6: FLOW, 250 SECOND-FEET (35 CUBIC FEET PER SECOND THROUGH PRIMER)										
Forebay	.....	.....	.....	.....	767.3	767.3	767.3	0	.....	.....
1	2.10	24.75	10.1	1.6	762.2	763.8	766.8	4.6	17.2	14.2
2	9.77	8.00	31.3	15.2	752.5	767.7	765.0	12.5	28.4	8.8
3	17.00	9.60	26.0	10.5	729.5	740.0	759.6	30.1	44.0	5.7
4	35.33	12.50	20.0	6.2	741.9	747.1	752.0	10.1	25.5	9.8
5	45.30	12.55	19.9	6.2	738.9	745.1	749.4	10.5	26.0	9.6
Outlet	68.25	12.42	20.1	6.3	739.9	746.2	746.2	6.3	20.1	12.4
SIPHON NO. 5: FLOW, 470 SECOND-FEET (60 CUBIC FEET PER SECOND THROUGH PRIMER)										
Forebay	.....	.....	.....	.....	767.6	767.6	767.6	0.	.....	.....
1	2.00	39.8	11.8	2.2	760.6	762.8	766.6	6.7	20.8	19.7
2	11.00	16.5	28.5	12.6	752.2	764.8	764.8	12.6	28.5	16.5
3	19.77	19.1	24.6	9.4	731.6	741.0	760.3	28.7	43.0	10.9
4	37.27	24.6	19.1	5.7	738.9	744.6	751.4	12.6	28.5	16.5
5	49.27	24.6	19.1	5.7	737.9	743.6	748.8	10.9	26.5	17.7
Outlet	72.83	24.2	19.4	5.9	741.2	747.1	747.1	5.9	19.4	24.2

The reason Line *A* does not drop continually is because the mass of moving water fills only a part of the barrel of the siphon, the remaining part being occupied by eddies; that is, cavitation exists within the siphon barrel. The elevation of the energy line at the inlet is that of the forebay level plus the atmospheric head of 33 ft; at the outlet, it is that of the crown of the barrel (open water surface) plus the exit velocity head, plus 33 ft. A straight line drawn between these points would not be seriously inaccurate, but the line marked, "Energy Line *B*" (Figs. 7, 8, and 9), is believed to represent the conditions more accurately.

The differences between the elevations of Line *B* and those of the pressure line are the velocity heads from which the velocity of the mass of flowing water is obtained. Dividing the flow by the velocities thus obtained gives the "Useful Area *a*" for each section (Column (11), Table 3). Maintaining the same width of barrel, the amount the actual areas are reduced to obtain "useful areas," is shown by hatching on Fig. 3.

The hatched areas represent the parts of the siphon barrels occupied by eddies. Showing them alternately on the crown and invert may not repre-

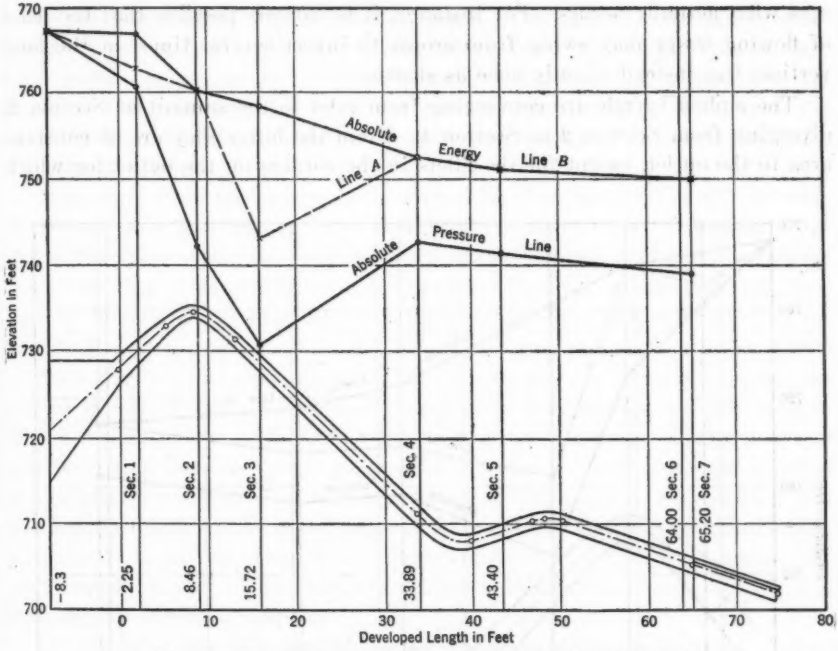

FIG. 7.—PRESSURE AND ENERGY GRADIENTS, SIPHON NO. 7.

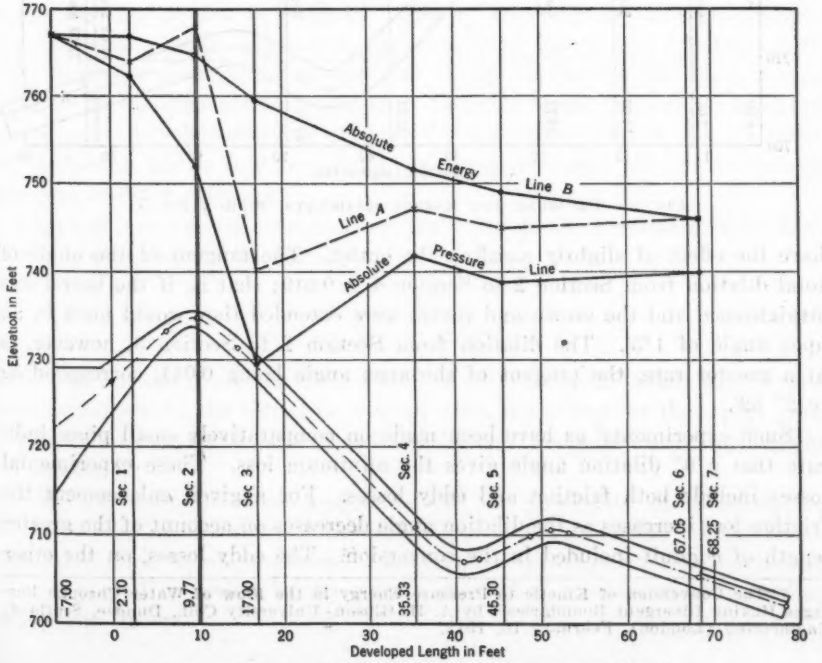

FIG. 8.—PRESSURE AND ENERGY GRADIENTS, SIPHON NO. 6.

sent what actually occurs. For instance, it is entirely possible that the mass of flowing water may swing from crown to invert several times in the long vertical leg, instead of only once as shown.

The siphon barrels are converging from inlet to the summit at Section 2, diverging from Section 2 to Section 4. From the latter they are of constant area to the outlet, except for the fillets in the corners on the outlet leg which

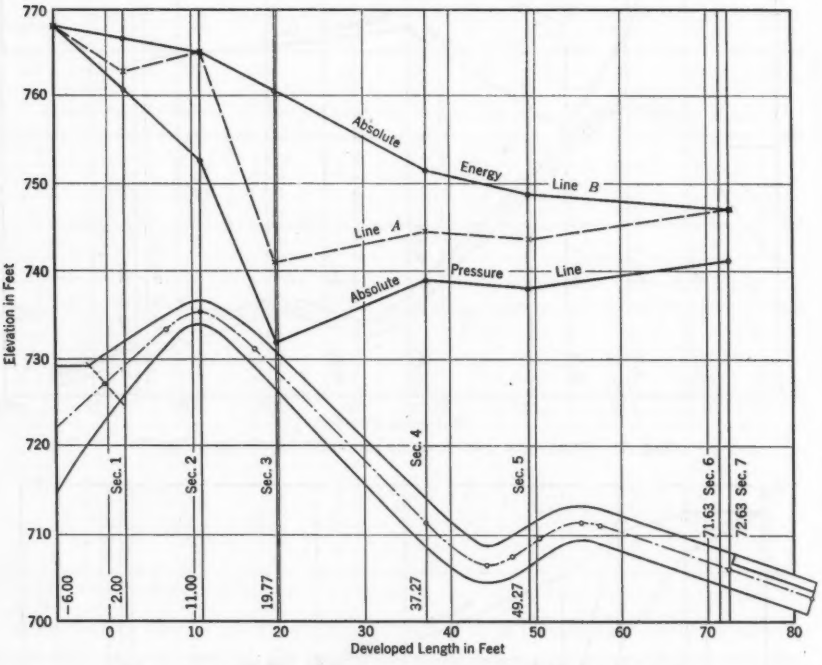

FIG. 9.—PRESSURE AND ENERGY GRADIENTS, SIPHON NO. 5.

have the effect of slightly nozzling the outlet. The tangent of the angle of total dilation from Section 2 to Section 4 is 0.019; that is, if the barrel was straightened and the crown and invert were extended they would meet in an apex angle of 1°5′. The dilation from Section 2 to Section 3, however, is at a greater rate, the tangent of the apex angle being 0.044, corresponding to 2° 32′.

Such experiments[3] as have been made on comparatively small pipes indicate that a 6° dilation angle gives the minimum loss. These experimental losses include both friction and eddy losses. For a given enlargement the friction loss increases as the dilation angle decreases on account of the greater length of conduit included in the conversion. The eddy losses, on the other

[3] "The Conversion of Kinetic to Pressure Energy in the Flow of Water Through Passages Having Divergent Boundaries," by A. H. Gibson, University Coll., Dundee, Scotland, *Engineering* (London), February 16, 1912.

hand, must diminish with the dilation angle. The sum of eddy and friction losses, therefore, has a minimum value which for such experiments as have been made appears to be between 6 and 10 degrees. If eddy losses are divorced from friction losses, however, it is practically certain that the smaller the apex angle the smaller the eddy loss, becoming zero simultaneously with the apex angle.

It appears from the foregoing data that the dilation angle of $2\frac{1}{2}°$ between Sections 2 and 3 is too great to maintain the barrel full of flowing water under the high velocities obtaining in these siphons. The mass flowing from Section 2 continued to contract, reaching its minimum section near Section 3. Thereafter, it gradually expanded independent of the dilation of the barrel.

That Section 3 was the one of maximum velocity was evidenced during the test of Siphon No. 6. Notice that the pressure head in Section 3 is practically one atmosphere. While the test was under way a small column of water was shot from Section 3 through about 15 ft of $\frac{1}{8}$-in. hose on to the high leg of the mercury column. This was puzzling at first until it was realized that the water column in the barrel separated momentarily under the nearly complete vacuum, and the impact of joining again was sufficient to produce the phenomenon noted. This was always noticeable when Siphon No. 6 was running, by rhythmic impacts.

At the inlet the flow conditions are complicated by the division of flow, part flowing in the barrel proper and part through the primer. The estimated division of flow is given in Table 3. On entering the siphon barrel the water flows nearly horizontal, particularly at the crown of Station 0.00. This doubtless prevents the barrel from filling. Deducting the hatched area (Fig. 3) leaves what probably is a more desirable shape. The water entering the primer joins that entering the barrel at the summit, which has the effect of producing eddies and, no doubt, accounts for some of the anomalous pressure readings obtained.

Siphons are usually compared on the basis of their coefficients of flow, which is expressed by the formula,

$$C = \frac{Q}{a\sqrt{2gH}} \qquad (1)$$

in which, a is the transverse area, and H is the total head. Some writers base this coefficient on the throat or summit area and others on the outlet area. Table 4 shows the coefficients obtained under both assumptions for the siphons in question.

The variation in these coefficients and the possibility of manipulating them by varying the areas of the siphon put emphasis on the necessity of a standardization of nomenclature. Before offering suggestions to accomplish this it is well to review briefly certain theoretical considerations regarding siphons in general.

TABLE 4.—COEFFICIENT OF FLOW

Description	Siphon No. 7	Siphon No. 6	Siphon No. 5
Flow, in cubic feet per second, Q	119	250	470
Area at summit, in square feet, a_2	3.43	8.00	16.5
Total head, in feet, H	28.4	27.4	26.4
$\sqrt{2gH}$, in feet per second, V_H	42.7	42.0	41.2
$a_2 V_H$, in cubic feet per second, Q_2	146	336	680
$\frac{Q}{Q_2} = C_2$	0.82	0.75	0.69
Area at outlet, in square feet, a_7	4.43	12.42	24.20
$a_7 V_H$, in cubic feet per second, Q_7	189	521	998
$\frac{Q}{Q_7} = C_7$	0.63	0.48	0.47

THEORETICAL ANALYSIS

The writer has used certain terms that should be defined before an intelligent discussion can be undertaken; namely:

Summit.—The highest transverse section of the barrel of the siphon at right angles to the direction of flow.

Invert.—The side of the barrel corresponding to the bottom at the summit.

Crown.—The side opposite the invert.

Static Head.—The water head on any point in the barrel of the siphon; that is, the difference in elevations of the point in question and the water surface at the inlet.

Atmospheric Head.—The head of water corresponding to a pressure of one atmosphere (33 ft at Leaburg).

Absolute Head.—The static head plus the atmospheric head.

Siphon Head.—The static head on the water surface at the outlet; that is, the difference in elevations of the water surfaces at inlet and outlet.

Absolute Pressure Head.—If from the absolute head at each point in the siphon there is deducted the energy losses to that point and the velocity head at the point, the remainder is the absolute pressure head on that point. It is the pressure shown by a piezometer at the point in question.

Absolute Pressure Line.—A line joining the absolute pressure heads for all points in the siphon barrel—also called the hydraulic grade line, a line joining the pressures as indicated by piezometers along the siphon barrel.

Energy Head.—The pressure head plus the velocity head at any point.

Energy Line.—A line joining the energy heads at each point in the siphon barrel.

Submergence.—The amount the summit section at its crown is drowned by water in the down-stream leg.

Let H = the siphon head as herein defined; m, the atmospheric head; E_{12}, the energy losses between Sections 1 and 2; E_{34}, losses between Sections 3 and 4, etc.; h, the velocity head; h_1 at Section (1), h_2 at Section (2), etc.; d, the difference in elevation of the inlet water surface and crown at the summit; and s, the depth the crown at the summit is submerged by water in the down-stream leg.

The energy losses in a siphon consist of those due to the entrance, conduit friction, and curvature. Each loss, and the total, may be expressed in terms of flow. The maximum velocity obtains at any point when the "absolute head" (as defined) is entirely consumed in supplying the preceding energy losses and the velocity head at that point. A Venturi section may be installed at any point within a siphon barrel and the entire static head plus one atmosphere, may be utilized on its throat, in producing velocity at, and providing for, the losses up to the throat. At such points of maximum velocity the "absolute pressure head" is zero.

It will be convenient to consider a siphon in two parts: (1) The up-stream leg from inlet to summit; and (2) the down-stream leg from the summit to the outlet. Fig. 10 shows such a siphon. The outlet is submerged and the

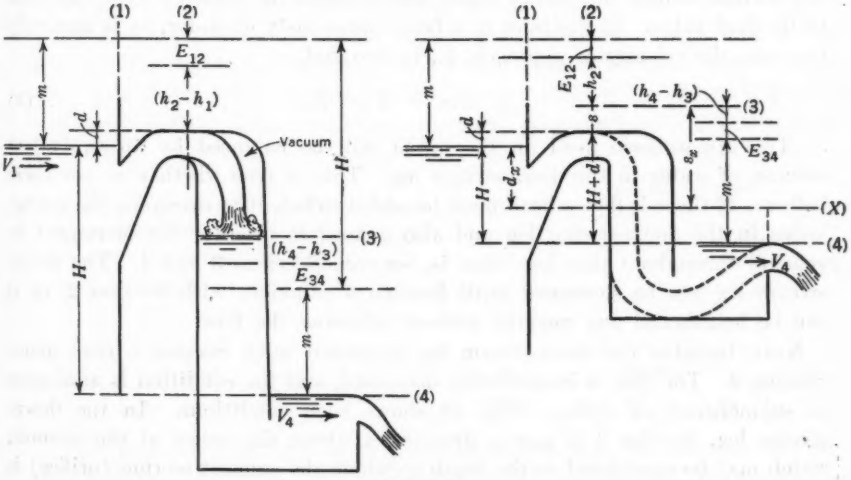

FIG. 10.

down-stream leg is long enough to maintain a complete vacuum at the summit. (A "complete vacuum" is impossible, of course. The kind of vacuum that is meant is one that would exist under the following circumstances: Imagine a pipe, say, 36 ft long, with one end sealed, the other ending in a valve. Fill the pipe with water and close the valve. Place the pipe vertical with the valve submerged 1 ft in a tank of water; then open the valve. The water in the pipe will fall about 1 ft, leaving a foot of "complete vacuum" at the top of the pipe. The absolute pressure in the top of the pipe will be the vapor pressure plus some pressure from air released from the water.)

The down-stream leg is large in Fig. 10(*a*) so as to show the jet action of the water passing the summit. It is obvious that the velocity of the water will continue to increase under the action of gravity until it strikes the water at Section 3. If the velocity increases the transverse area must contract, so that the maximum velocity does not necessarily exist at the summit section. If the down-stream leg were shortened until Sections 3 and 2 coincided, the

jet action would still exist, the velocity would increase, and the area would contract as long as the retarding force of water friction on the jet was less than the accelerating force of gravity. This is the phenomenon that was actually observed in the three Leaburg siphons. Figs. 7, 8, and 9 show that at Section 3 in each siphon practically the entire atmospheric head was consumed in overcoming losses and in producing velocity.

The condition shown in Fig. 10(*a*) will produce the maximum flow in a siphon. The entire absolute head on the summit is utilized in supplying the energy losses and producing the velocity through the summit section; thus,

$$m = d + (h_2 - h_1) + E_{12} \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

That is, the atmospheric head must raise the water to the crown of the summit section, supply the energy losses, and increase the velocity from its initial to its final value. If the inlet is a fairly large body of water, as is generally the case, the velocity of approach, h_1, is zero and,

$$h_2 = m - d - E_{12} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The atmospheric head on the outlet will be balanced by an equivalent column of water in the down-stream leg. This is true whether or not there is flow. If there is flow a head must be added sufficient to overcome the energy losses in the down-stream leg and also a head to produce the increment in velocity throughout that leg—that is, between Sections 3 and 4. The down-stream leg can be shortened until Section 3 coincides with Section 2, or it can be lengthened any amount without affecting the flow.

Next, imagine the down-stream leg shortened until Section 3 rises above Section 2. The flow is immediately decreased, and the condition is analogous to submerging an orifice. Fig. 10 shows such conditions. In the down-stream leg, Section 3 is now a distance, s, above the crown at the summit, which may be considered as the depth to which the summit section (orifice) is submerged. The summit section now discharges into something less than a vacuum and the flow correspondingly diminishes. In the down-stream leg,

$$s = m + E_{34} + (h_4 - h_3) - H - d \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The velocity head at the summit is then obtained from Equation (3) modified thus,

$$h_2 = m - d - E_{12} - s = H - E_{14} - (h_4 - h_3) \ldots\ldots\ldots\ldots (5)$$

The exit velocity from the up-stream leg now becomes the initial velocity for the down-stream leg, and $h_2 = h_3$. Hence Equation (5) reduces to,

$$h_4 = H - E_{14} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

That is, the velocity head at the siphon outlet is the siphon head less all energy losses. This is perfectly obvious.

The down-stream leg may be modified, as shown by the dotted lines in Fig. 10(*b*), without loss of generality. In the vertical down-stream leg the

velocity will increase until it reaches a maximum with a corresponding contraction of the area of the jet. The point of maximum velocity will be where the accelerating and retarding forces balance. This point appears very difficult, if not impossible, of determination by mathematical analysis and is greatly complicated by the fact that the weight of water diminishes in proportion to its acceleration.

Referring again to Equations (5) and (6), it is noted that the velocity head at the summit section disappears; that is, the velocity in any section between inlet and outlet is indeterminate by such analysis.

The only manner in which the velocity at any point between inlet and outlet of the siphon barrel may be determined is by measuring it directly, by calculation from measured pressure head, or by assuming that the barrel is running full at that point, and dividing flow by area.

Data on the losses in curves of such conduits are very meager and unsatisfactory. The losses in Siphons Nos. 7 and 6 were calculated using available curve loss data,[4] while friction losses were computed by Kutter's formula with $n = 0.013$. The comparison between calculated and observed losses follow:

	Siphon No. 7	Siphon No. 6
Calculated:		
Entrance loss	1.0	0.6
Friction loss	9.0	3.2
Curve loss	14.6	11.6
Total	24.6	15.4
Loss by tests	17.2	21.1

The necessary assumption in the calculations that the barrels were running full was, of course, not an actuality, which may account for the anomalous results. The calculated losses for Siphon No. 7 were greater and those for Siphon No. 6 were less than the observed.

Coefficient of Flow

The generally accepted coefficient of flow is the ratio of the actual to theoretical flow. Where the transverse areas of the siphon barrels are not uniform, there has been some confusion as to which area should be used to determine the theoretical flow. This coefficient for the outlet area is written:

$$C_4 = \frac{Q}{a_4 \sqrt{2gH}} \qquad (7)$$

but, by Equation (6), $H = h^4 + E_{14}$ (notation as in Fig. 10), and,

$$h_4 = \frac{V^2_4}{2g} = \frac{Q^2}{a^2_4 2g} \qquad (8)$$

[4] King's Handbook of Hydraulics, Second Edition, p. 193.

The energy loss, E_{14}, is the sum of all losses through the siphon, consisting essentially of entrance, friction, and bend losses. Each of these losses, and their sum, may be put in terms of flow having the form:

$$E_{14} = \frac{B Q^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which, B is a composite constant.

Substituting Equations (6), (8), and (9), in Equation (7),

$$C_4 = \frac{1}{\sqrt{1 + a_4^2 B}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

This flow coefficient is theoretically constant for a given siphon and is independent of the head on it or the flow through it (provided there is no appreciable amount of entrained air), since a and B are purely dimensional quantities. They can never be zero and, therefore, the coefficient for the outlet can never exceed unity. The factor, B, varies inversely as areas and directly as lengths. The coefficient, C, therefore, varies inversely as lengths and directly as all areas, except the outlet area for which it varies inversely.

The designer may diminish the coefficient by expanding the outlet, provided he can keep it full, at the same time increasing the flow and thereby increasing the efficiency of the siphon. Conversely, he may increase the coefficient by nozzling the outlet while he diminishes its flow and reduces the siphon's efficiency. If he keeps the outlet constant, he can increase the coefficient by increasing any or all areas above the outlet, or he can decrease it by lengthening the siphon barrel.

If the coefficient of flow be based on the summit area, the minimum area, or any other area, Equation (10) becomes,

$$C_x = \frac{a_4}{a_x} \frac{1}{\sqrt{1 + a_4^2 B}} = \frac{a_4}{a_x} C_4 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

If a_x is less than $C_4 a_4$, then C_x becomes greater than unity. This can only happen, however, when the velocity due to the absolute head at any point is expressed in terms of the siphon head. For example, if a Venturi section with a throat area of 2.0 sq ft was put in Siphon No. 7 at Section 4, the flow would be reduced to 107.0 sec-ft, while the theoretical flow would be 85.4 sec-ft, resulting in a coefficient for the Venturi throat area of 1.26. The absolute pressure head at Section 4 that is used up in producing velocity through the throat, is 44.5 ft, of which, 21.1 ft is atmospheric. The siphon head is 28.4 ft. There appears to be no advantage in using other than the outlet area for computing the coefficient of flow. The writer suggests that the definition of the coefficient of flow for a siphon be adopted as the ratio of actual flow through the siphon to theoretical flow through its outlet area.

It is interesting to note that the coefficient of flow for any transverse section is also that for a unit area at that section; that is, it is the ratio of actual to theoretical velocity, since,

$$C_x = \frac{Q}{a_x \sqrt{2gH}} = \frac{V_x}{V_H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

It is obvious that a coefficient computed for a section that is not running full is meaningless.

If the water contains appreciable quantities of entrained air the coefficient will be reduced in proportion to the reduced flow. This is the most prevalent source of variable coefficients for the same siphon. If there is no entrained air the coefficient should remain constant for a given siphon regardless of variations in head and flow.

Efficiency of a Siphon

The efficiency of a siphon is not measured by its coefficient of flow because the designer can vary that coefficient by changing the dimensions of the siphon and can produce opposite variations in the efficiency. Consider, for a moment, the siphon as a machine. The efficiency as ordinarily conceived is the ratio of output energy to input energy. This is equivalent to the ratio of the outlet velocity head to the siphon head, that is (see Equation (12)),

$$\frac{h_4}{H} = \left[\frac{V_4}{V_H}\right]^2 = C_4^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Treated as a machine, therefore, the efficiency of a siphon would be merely the flow coefficient, squared. When referring to the efficiency of a pipe line, it is thought of as the ratio of the useful to the total head; that is, the ratio of $\frac{\text{Total head less losses}}{\text{Total head}}$, which is identical with Equation (13).

The purpose of a siphon is to discharge water through the utilization of atmospheric pressure. It would seem, therefore, that its efficiency is best measured by the ratio of the actual to its theoretically maximum flow. Such maximum flow obtains when one atmosphere is entirely used in producing velocity through the summit section, and up-stream energy losses are not included. This definition would lead to the expression (see Fig. 10(*a*)),

$$\text{Efficiency} = \frac{Q}{a_2\sqrt{2gm}} = \frac{V_2}{V_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Siphons with low siphon heads would have low efficiencies and *vice versa*. A siphon with a long down-stream leg that contains a shear drop would have no greater efficiency than if the crown of the summit section were barely submerged. The efficiencies of the Leaburg siphons according to this criterion were, as follows:

	Siphon No. 7	Siphon No. 6	Siphon No. 5
Velocity at summit section from Table 3..	34.6	28.4	28.5
Velocity for one atmosphere.....	46.1	46.1	46.1
Efficiency, percentage.....	75	61.5	61.6

The writer proposes to define the efficiency of a siphon as the ratio of the actual velocity through the summit section to the velocity corresponding to one atmosphere.

Conclusion

In conclusion, it may be observed that some of the phenomena and behavior of the siphons herein outlined could never be detected in the laboratory by the use of models. In such models, velocities never reach values high enough to disclose the jet action noticeable in the large siphon. The siphon barrel of a laboratory model will always run full after the siphon has been thoroughly primed. The barrel of the prototype may never run full, throughout its entire length. Nevertheless, much valuable information can be obtained by tests on models of siphons, once their limitations are fully recognized.

Acknowledgments

In performing the tests described in this paper, the writer was assisted by Mr. H. H. Wheaton, Graduate Student at the University of California, now John Freeman Traveling Scholar in Europe, and by the Operating Staff at the Leaburg Plant; the late Carl A. Mc Clain, M. Am. Soc. C. E., Superintendent also rendered valuable assistance and co-operated fully in the work.

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PAPERS

USE AND CAPACITY OF CITY STREETS

By Hawley S. Simpson,[1] Assoc. M. Am. Soc. C. E.

Synopsis

City passenger transportation is provided by two distinct classes of vehicles; (*a*) Individual units, such as passenger automobiles and taxicabs; and (*b*) mass transportation vehicles, including street cars, motor buses, and (to a small extent) trolley buses. Subways and elevated lines are omitted from Classification (*b*) because they operate on private rights of way. The passenger-carrying ability, or passenger capacity, of vehicles of these classes varies within wide limits, and thus the utility of the street system, bears a direct relation to the type of unit used for passenger transportation.

In this paper particular attention is directed to the passenger capacity of city streets when used solely by passenger automobiles and when used jointly by private and public vehicles. Special consideration is given to the effect that parking has in reducing both vehicle and passenger capacity. Detailed tables are included showing vehicle and passenger capacities of streets of different widths when parking is allowed and when it is prohibited. These tables have been prepared after a study of a large number of theoretical analyses and field surveys of actual street conditions and capacities, and are believed to provide a reasonably reliable measuring stick of potential street capacities. It will be found, in many cases, that street usage on thoroughfares commonly considered to be "congested," falls far short of the limits here developed, for congestion more often occurs because of deficient traffic control and regulation than because of traffic saturation.

Improvement in city traffic conditions can result only when careful attention is given to the relative economic importance and efficiency of both private and public vehicles. Street improvement projects, as well as traffic regulation plans, should be developed so as to provide the highest possible facility of movement for both private and public vehicles. In this way the services

Note.—Discussion on this paper will be closed in November, 1933, *Proceedings*.

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rendered by each may be developed to fill the public need for transportation most satisfactorily.

The study naturally shows very much greater passenger capacities on streets served by mass transportation vehicles, despite the fact that the total vehicle capacity suffers a reduction. This merits the serious consideration of public authorities faced with the necessity of augmenting street facilities.

Introduction

In the modern scheme of city life the transportation problem takes foremost rank. None is more difficult to treat to the satisfaction of all, because none, perhaps, so closely touches every phase of social, commercial, and industrial activity.

The basic pattern of local transportation is built around the so-called "necessity" travel—from home to work in the morning and again from work to home in the evening. To this are added the incidental movements of traffic upon which the efficient functioning of every metropolitan area depends. All these matters are in a continual state of flux. Habits and places of living, work, and play are constantly changing. Decentralization of homes and of commercial and industrial establishments has been taking place for some time. Simultaneously, in some places, there has been a noticeable tendency toward recentralization; many individuals now find it more convenient to live near the center of activity, and certain types of businesses are recognizing new advantages in crowded areas. Social and recreational demands of the community produce their own transportation problems. In this respect, too, change is evident. Simple home, or neighborhood, amusements no longer suffice. Young and old alike seek outside entertainment more frequently and at greater distances from the home, while travel, as such, is becoming an important means of recreation.

If the transportation system of a city—embracing streets and public and private facilities for travel—is properly conceived the nature and direction of city growth may be moulded in ways calculated to promote the best interests of the community. Improperly organized, growth may be stifled or may take place when and where the streets are inadequate for all essential requirements, with corresponding economic loss. Knowledge of the potential ability of city thoroughfares to serve the public needs is essential, therefore, to all who are in any way concerned with the administration of municipal affairs.

This paper has been prepared in the course of studies that necessitated the collection of many widely scattered data on the subject of street use and street capacity. It has not been possible to conduct the extensive research necessary to lift the subject from the realm of controversy; almost complete reliance has had to be placed upon existing surveys. The insufficiency of this information will be readily apparent to the reader and, it is hoped, may serve to encourage further study of the problems involved in moving passengers and merchandise through city streets with particular attention to the capacity characteristics of public ways.

Street capacity is usually measured in numbers of vehicles passing a given point in some unit of time—usually 1 hr. Vehicles, however, are only the instruments of transportation, the primary function of a street system being to provide for the movement of persons and merchandise and to give public access to abutting private property which thus gains value by becoming accessible to those who wish to live, work, shop, play, or engage in commerce or industry thereon. Land, in itself, has little inherent worth. Richard M. Hurd has stated:[2]

"In cities, economic rent is based on superiority of location only. * * * thus the value of all urban land ranges from that which least serves the smallest number of people of the lowest economic quality, up to that which best serves the largest number of people of the highest economic quality."

On this account the passenger capacity of a city street is of much greater social importance than its vehicle capacity. Therefore, in this paper, attention is directed primarily to the passenger capacity of streets, beginning with an analysis of their vehicle capacities.

The Meaning of Street "Capacity"

"Capacity" is a difficult word to define when used with reference to a street system. It conveys an implication that there is some definite limit to the number of vehicles that may move along a street in a stated period of time. No such rigid interpretation is possible. Were the carrying ability of a street to be taxed to its ultimate—in other words, were its theoretical maximum capacity to be reached—conditions would result which would not long be tolerated. Assuming that such a condition should occur on some one street, motorists would be inclined to divert to near-by, less direct, and more lightly traveled highways until a condition of stability was reached, with the more favorably located streets somewhat more densely used than the others. Under such conditions it might be said that the streets were carrying capacity loads; yet each might be capable of accommodating considerable additional traffic if it were to present itself and if no convenient alternative streets were available.

Obviously, then, the capacity of a street is not capable of accurate measurement. For present purposes, it will be sufficient to state the sense in which the word, "capacity," is to be used:

"The capacity of a city street is reached when the volume of traffic is so great that any further increase will result in conditions of movement so unsatisfactory to the users that less favorably situated routes are preferred by a considerable proportion of those who would normally find the original street most useful."

It is obvious, from this statement, that the "capacity" of any particular street, as the word is here used, will be largely dependent upon the relative utility of, and volume of traffic upon, near-by streets. Thus, the traffic capacity of the "Fifth Avenues" of large cities will be considerably higher, generally, than that of the "Main Streets" of smaller cities. By confining investigations to conditions as they exist in very large cities, the results may be

[2] "Principles of City Land Values," by Richard M. Hurd, 1905, pp. 1 and 13.

considered as most applicable to streets that are commonly considered "congested," and indicative, therefore, of reasonable limiting conditions consistent with satisfactory speed and comfort of travel.

Vehicle Capacity of Highways Without Grade Crossings

The number of vehicles per lane which will pass a given point in unit time is a function of their average speed and center-to-center spacing, as expressed in the equation:

$$N = 5\,280\left(\frac{V}{P}\right) \qquad (1)$$

in which, N is the number of vehicles per lane per hour; V, the velocity of vehicles, in miles per hour; and P, the average center-to-center spacing of vehicles, in feet.

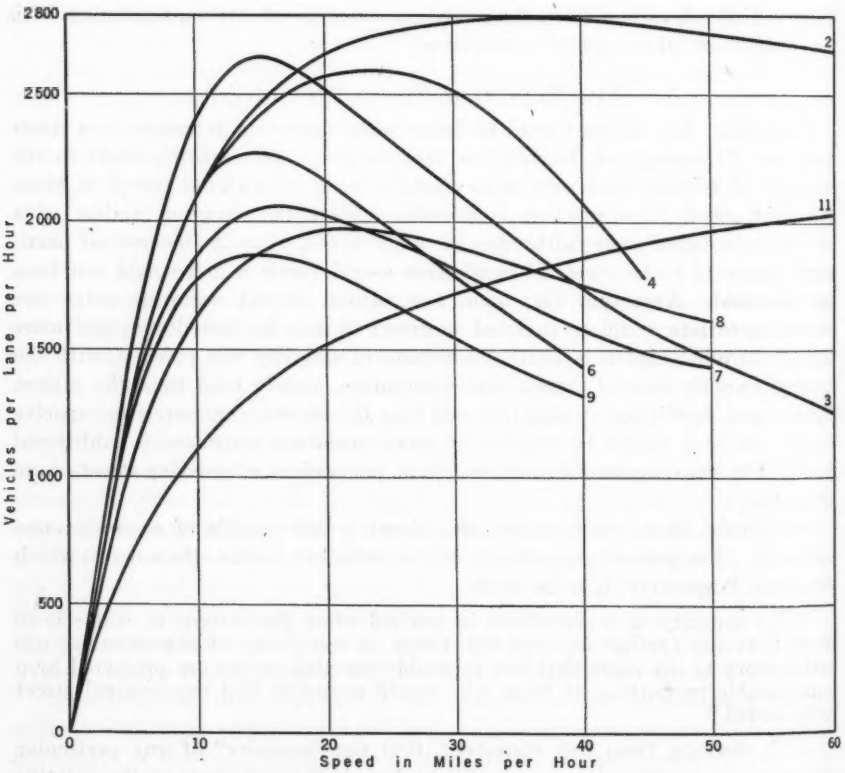

Fig. 1.—Various Determinations of the Hourly Non-Stop Capacity of a Single Lane of Vehicles (See Text for Sources of Data).

Many attempts have been made to determine, through theoretical studies and by actual observation, the capacity of a single lane of vehicles uninterrupted by cross-traffic. Fig. 1 presents several of the curves, and Table 1

shows the computed maximum capacity and the speed at which each is estimated to occur. Maxima determined by others, and not recorded on Fig. 1, are also included as Item Nos. 1, 5, and 10, in Table 1.

TABLE 1.—VARIOUS DETERMINATIONS OF THE HOURLY NON-STOP CAPACITY OF A SINGLE LANE OF VEHICLES.

Item No. (see Fig. 1)	Capacity (maximum vehicles per hour)	Speeds, in miles per hour	Spacing, from center to center, in feet	Conditions
1.....	3 320	20.8	33	Minimum spacing of individual vehicles.
2.....	2 800	34.5	65	.....
3.....	2 640	15.	30	.....
4.....	2 600	23.5	48	.....
5.....	2 430	20.7	45	Average group spacings.
6.....	2 264	15	35	"Emergency stop" spacing, 50% four-wheel brakes and 50% two-wheel brakes.
7.....	2 045	16.5	43	Four-wheel brakes on all vehicles.
8.....	1 969	22	59	"Ideal" conditions.
9.....	1 880	15	42	Average group spacings.
10.....	1 852	24.9	71	Average spacing of all vehicles.
11.....	.....	....	..	No maximum reached (see Fig. 1).

The information contained in Table 1 was obtained from various sources, as follows: Items Nos. 1, 5, and 10 were reported by R. F. Kelker, Jr.,[3] M. Am. Soc. C. E.; Items Nos. 2 and 3, by A. N. Johnson,[4] M. Am. Soc. C. E.; Item No. 4, by the Regional Planning Commission, County of Los Angeles, California,[5]; Items No. 6,[6] No. 7,[7] and No. 8,[8] by the Committee on Highway Traffic Analysis, Highway Research Board; Item No. 9, by Harold M. Lewis and E. P. Goodrich,[9] Members, Am. Soc. C. E.; and Item No. 11, by Sigvald Johannesson,[10] M. Am. Soc. C. E. The data for Curve 11, Fig. 1, are not given in Table 1 (see Item No. 11), because no maximum is reached regardless of speed.

The wide variation in the results shown in Fig. 1 seems to confirm the statement that maximum street capacity can not be accurately ascertained. Nevertheless, surveys in several places show rather comparable results of actual performance, indicating that when the traffic volume has reached a point at which conditions are generally unsatisfactory, a proportion of the drivers seek other thoroughfares. The results of representative surveys appear in Table 2, for which the following authorities may be recorded: Item No. 1, by A. N. Johnson,[11] M. Am. Soc. C. E.; Items Nos. 2 and 3, by the American Transit Association[12]; Item No. 4, by the Highway Research Board, Committee on Highway Traffic Analysis[13]; Item No. 5, by the Cleveland Railway

[3] *Journal*, Western Soc. of Engrs., April 19, 1926, pp. 140–147.
[4] *Proceedings*, Eighth Annual Meeting, Highway Research Board, 1929, p. 115.
[5] *Proceedings*, Thirty-sixth Annual Convention, Am. Soc. of Municipal Engrs., 1931, p. 47.
[6] *Proceedings*, Seventh Annual Meeting, Highway Research Board, 1928, p. 233.
[7] *Proceedings*, Ninth Annual Meeting, Highway Research Board, 1930, p. 106.
[8] *Proceedings*, Seventh Annual Meeting, Highway Research Board, 1928, p. 231.
[9] Regional Plan of New York and Its Environs, "Highway Traffic," 1927, p. 86.
[10] "Highway Economics," McGraw-Hill Co., 1931, p. 91.
[11] *Proceedings*, Eighth Annual Meeting, Highway Research Board, 1929, p. 108.
[12] Rept. of Comm. on Street Traffic Economics, Am. Elec. Ry. Assoc. (now Am. Transit Assoc.), 1928, p. 69.
[13] *Proceedings*, Eighth Annual Meeting, Highway Research Board, 1929, p. 92.

Company[14]; Item No. 6, by the Detroit, Mich., Rapid Transit Commission[15]; Item No. 7, by Ole Singstad, M. Am. Soc. C. E.; Item No. 8, by E. A. Byrne,[16] M. Am. Soc. C. E.; and, Item No. 9, by Day and Zimmerman, Inc.[17] Referring to Item No. 7, the capacity of the Holland Tunnel is computed by

TABLE 2.—MAXIMUM OBSERVED RATES OF TRAFFIC FLOW PER AVERAGE LANE—ONE DIRECTION.

Item No. (1)	Location (2)	Number of lanes in each direction (3)	Maximum Hourly Rates						Estimated capacity per lane per hour † (10)
			20 sec. (4)	1 min. (5)	5 min. (6)	10 min. (7)	30 min. (8)	60 min. (9)	
1	Baltimore-Washington Highway	1	.....	.....	1 968	.....	.....	1 502	1 500
	Lake Shore Drive, Chicago, Ill.:								
2	Average for both lanes	2	2 070	1 830	.....	1 392	1 365	1 349	1 335
3	Inner lane	2	2 520	2 160	.....	1 542	1 412	1 395	.....
	Superior-Detroit High-Level Bridge, Cleveland, Ohio:								
4	1928	3	.....	.....	.....	.....	.....	1 241	1 170
5	1927	2½*	.....	.....	1 958	1 874	1 699	1 557	1 230
6	East Grand Boulevard, Detroit, Mich	1	.....	1 800	.....	1 404	1 200	.....	.....
	New York, N. Y.:								
7	Holland Tunnel	2	.....	.....	.....	.....	.....	1 253	1 335
8	Queensborough Bridge	3	.....	.....	.....	.....	.....	1 482	1 470
9	Manhattan Bridge	2	.....	.....	.....	.....	.....	1 300	1 335

* Estimated number of effective lanes.
† Calculated by using the percentages of Table 3, multiplied by 1 500 (the estimated hourly non-stop capacity of a single lane).

the Engineers of the Port of New York Authority as 1 270 vehicles per hr per lane, in two lanes, when most of the traffic is composed of passenger vehicles. M. O. Eldridge, Assoc. M. Am. Soc. C. E., has stated[18] that delays incident to the purchase and collection of tickets, the weaving of vehicles from several lanes into two at the tunnel entrances, and the grades within the tunnel limit the capacity of the tubes.

In none of these surveys can it be stated definitely that non-interrupted traffic was observed or that sufficient traffic existed to fill the street completely. Photographs taken during the heaviest 20-sec flow on the Lake Shore Drive, in Chicago, Ill., showed material gaps in both lanes which could have accommodated other vehicles (if the supply had existed) at the rate of about one for every three observed. (See Table 2, Item No. 2.) If this possible density of use had occurred, the rate of flow would have been about 2 800 vehicles per lane per hr, being only about 500 vehicles per hr less than found in the observations by Mr. Kelker, based upon minimum spacings of individual vehicles, and checking exactly with the figure developed by Dean Johnson. (See Table 1, Items Nos. 1 and 2.)

[14] Survey Rept. on Superior-Detroit High-Level Bridge, June 18, 1927.
[15] Rept. on Vehicular Traffic in the Business District of Detroit, 1924, p. 16.
[16] *Civil Engineering*, July, 1931, pp. 934-935.
[17] Rept. on Highway Traffic Conditions and Proposed Traffic Relief Measures for the City of New York, 1929, p. 163.
[18] Rept. on Traffic Capacity of Holland Tunnel, 1931 Convention, Am. Road Builders Assoc.

Perhaps the ultimate capacity of a lane, for very short periods, may approach 3 600 vehicles per hr. At this rate, and at speeds of 22 miles per hr, observed in the Chicago case (Items Nos. 2 and 3, Table 2), the center-to-center spacing of vehicles would be about 32 ft. This does not seem unreasonable as the limiting figure. No such capacity figure should be used for design purposes, however. Maximum short-period traffic flows, as seen from Table 2, are almost twice as high as the hourly flow. A rate of flow for short periods as high as the probable ultimate will occur when the hourly traffic is about 1 800 vehicles per lane. The acceptance of such a figure for working purposes would leave only a small factor of safety, however. For present purposes a value of 1 500 vehicles per single lane per hr may be adopted as representing "capacity," with the realization that higher flows are possible and in the belief that the factor of safety allowed is no greater than is necessary to maintain reasonably fluid movement at speeds consistent with safety and efficiency.

Multiple-Lane Highways Without Grade Crossings

The value, 1 500 vehicles per single lane per hr, for lane capacity is predicated upon a single moving lane in each direction. On wider streets vehicles seldom travel in well-defined paths, but weave from lane to lane and straddle the imaginary line between lanes, thus occupying space potentially capable of accommodating two or more vehicles and reducing the street capacity. In order to overcome this condition, rigid restriction of each vehicle to its original lane is being practiced in a few instances. In the Holland Tunnel, between New York City and Jersey City, N. J., police patrols effectively prevent motorists from crossing the painted strip separating the two lines in each one-way tube. On the new upper level of the Queensborough Bridge, in New York City, an attempt was made to control the alignment of automobiles by constructing tracks for three lines of traffic in one direction.[16] This resulted in lanes which seemed unsafe and at the protest of motorists the tracks were removed so that the roadway now (1933) handles only two lanes.

The effect of weaving and straddling on non-stop highways has been investigated by Mr. Kelker.[3] The results are shown in Table 3. From this it appears that an express highway with more than three moving lanes in each

TABLE 3.—Relative Capacity of Multiple-Lane Streets, Assuming the Single Lane as 100 Per Cent

Number of lanes in one direction	Efficiency per lane, percentage	Capacity of half street, percentage	Number of lanes in one direction	Efficiency per lane, percentage	Capacity of half street, percentage
1	100*	100	4	65‡	260
2	89†	178	5	52‡	260
3	78†	234	.....	...	...

*Assumed. † Actual.

direction (a total of six lanes) is of doubtful economy. The construction of the fourth lane, adding 33⅓% to the width, increases the capacity of the street only about 11%, while a street with five lanes in each direction has no greater capacity than one of four lanes.

Using the data of Table 3 and the estimated capacity of a single lane (1 500 vehicles per hr), the capacities of grade-separated highways of from one to three lanes have been computed (Table 2, Column (10)) and compared with the observed maxima of Table 2, Column (9). In studying this comparison it should be borne in mind that the estimated capacities have been adopted with a view to allowing a reasonable factor of safety, and it is to be expected that at times they will be exceeded.

Vehicle Capacity of Surface Streets

Uninterrupted traffic flow is possible on only a few of the present highways. Surface street traffic is subject to frequent interruption by crossing vehicles or by "stop-and go" signals. The capacity of any single street could be approximated if the timing and spacing of signals, or the volume of cross-traffic, or both, were known, and if speed of vehicles and other pertinent factors were available; but such computations are of practical value only in specific instances. For the purpose of this paper it will be assumed that the capacity of a traffic lane devoted to straight-through movement on a surface street is equal to 60% of its non-stop capacity, namely, 900 vehicles per lane per hr. This applies, of course, only to major streets on which the traffic volume is approximately twice as great as that on the cross-streets. Traffic volumes exceeding 900 vehicles per lane per hr have been observed on streets with traffic signals and with grade crossings; but such streets are not the average that must be dealt with in general observations.

Factors Affecting Capacity of Multiple-Lane Surface Streets

The capacity of multiple-lane surface streets cannot be determined in the same manner as in the case of grade-separated highways. Weaving and straddling doubtless reduce street capacities somewhat, but they are probably not very important except on unusually wide surface highways, as for example, streets of more than eight moving lanes. Other factors, not common to non-stop highways, affect surface street capacities so greatly that the relative importance of weaving and straddling is almost negligible on streets of average widths. Some of the major factors that require consideration are: Turning movements; parking; street car traffic; and bus traffic.

Turning Movements.—Left-hand and right-hand turns reduce the capacity of the lanes from which each is made by delaying the traffic (behind the turning vehicle) during the maneuver. To determine accurately the effect of left-hand turns upon capacities, the traffic flow and incidence at each intersection should be known for traffic in the opposite direction as well as for the traffic that turns. Pedestrian volumes and direction would also be essential.

In the case of right-hand turns, the volume of turning traffic and the pedestrian traffic in the original direction of the turning vehicle should be known. There being no recorded observations of such character it has been assumed that the delay to traffic in turning lanes will average 15% of its moving time by left turns and 5% by right turns. If a single lane must accommodate through traffic as well as both right and left turns, the reduction in capacity would be the sum of these percentages.

Parking.—Curb parking eliminates one entire traffic lane and probably somewhat lowers the capacity of the lane nearest thereto because of double parking and conflict between vehicles moving to and from the curb lane. The exact measure of this factor would depend upon the prevalence of double parking and the frequency of parking line interchange. For present purposes it will be sufficient to assume, in general, that the capacity of the lane next to the parked lane is reduced 5% under ordinary regulations permitting parking.

If parking is not allowed the curb lane nominally becomes available for moving traffic, but since stopping for the discharge of passengers and merchandise is usually permitted, the capacity of the curb lane for moving traffic may vary between wide limits, depending upon the extent of the desire or need to stop. Observations in Chicago have shown traffic flows in the curb lane on central district streets exceeding a rate of 600 vehicles per hr during the maximum half-hour, when parking is prohibited.[19] The results of similar observations in Detroit, Mich., are shown in Table 4.[20] Referring to Item No. 4, the third or center lane was used by 51 street cars and 634 automobiles. On Jefferson Avenue (Item No. 5, Table 4) the fourth or center lane was used by 41 street cars, 25 buses, and 25 automobiles. Therefore, in these two items, the last, or center, lanes were not included in computing values in Column (5) of the table.

TABLE 4.—USAGE OF CURB LANE DURING MAXIMUM HOUR, IN DETROIT MICHIGAN (PARKING PROHIBITED).

Item No. (1)	Location (2)	Number of one-way free lanes (3)	Vehicles per hour in curb lane, parking prohibited (4)	Curb-lane vehicles as percentage of average vehicles per lane in adjacent lanes (5)
1	John R. Street, at East Grand Boulevard.....	2	384	70
2	West Grand Boulevard, at 14th Street.....	3	295	40
3	West Grand Boulevard, at Cass Avenue.....	4	543	106
4	Grand River Avenue, at West Grand Boulevard	3	830	93
5	Jefferson Avenue, at East Grand Boulevard...	4	707	78

On the basis of these observations it seems conservative to assume that the average curb lane, when parking is prohibited, and allowing for the influence of right turns, is the equivalent of at least one-half a free lane—450 vehicles per hr. If left turns must be made from this lane its effectiveness will be reduced 15% as previously stated.

Effect of Street Cars on Automobile Capacity.—Obviously, there must be some relation between the motor-vehicle capacity of a street-car lane and the number of street cars operated on it. If the tracks are used by only a small number of street cars, motor vehicles should be able to use that lane with practically the same freedom as the adjoining free lane. If, on the other

[19] Rept., Committee on Street Traffic Economics, Am. Elec. Ry. Assoc. (now Am. Transit Assoc.), 1930, p. 59.

[20] "Vehicular Traffic in 1930", Detroit Rapid Transit Comm., 1930, p. 75.

hand, street-car headways are very short, so that the maximum capacity of the track in street cars per hour is reached or approached, efficient operation of street cars implies a minimum of motor vehicles in the street-car lane. Such conditions may hold, however, for only a short time in the rush period while during the remainder of the day motor-vehicle operation on the car tracks may not greatly hinder street cars. Data pertaining to this subject have been secured from Detroit[21] and Chicago[22] and are given in Table 5.

TABLE 5.—OBSERVED HOURLY RATE OF TRAFFIC FLOW IN STREET CAR AND ADJACENT FREE LANES*

City and street	Period, in minutes	In Street-Car Lane per Hour		Automobiles per lane in adjacent free lanes per hour
		Street cars	Automobiles	
Chicago, Ill.:				
Broadway	10	60	568	270§
Broadway	30	34	506	212§
Broadway	60	34	432	205§
Broadway	60	23	471	281§
Broadway	60	36	265	323§
Broadway	60	30	618	567§
Broadway	60	30	275	299§
Broadway	60	27	385	210§
Washington Boulevard	30	108	282	563‖
Crawford Avenue	60	60	347	¶
Crawford Avenue	60	60	357	¶
Madison Street	60	36†	593	¶
Detroit, Mich.:				
Jefferson Avenue	60	60‡	270	766‖
Clifford Street	60	32‡	400	700§
Gratiot Avenue	60	118‡	191	545§
Grand River Avenue	60	51	634	860‖

* There are no data as to whether capacity was approached in any of these instances.
† Two-car trains.
‡ Not reported in Detroit survey.[21] Data furnished by J. P. Hallihan, M. Am. Soc. C. E.
§ In one free lane.
‖ Average of two free lanes.
¶ Not recorded.

Despite the comparative meagerness of these data and the irregularity of the Chicago figures, where automobiles frequently use the car-track lane more intensively than the free lane, automobile usage seems to bear some relation to street-car frequency. The Chicago data indicate relatively free movement in the street-car lane; otherwise, more automobiles would have operated in the free lanes. The heavier flows in the car lane indicate that there may be a safety advantage in driving in the track area in comparison with the lane next to a line of parked cars.

About the best that can be deduced from these observations is the general conclusion that the operation of 120 street cars, or more, per hr, in one direction, is sufficient to require full use of the car-track area and that each reduction of 20 cars per hr below this figure will permit reasonably free movement space for 125 automobiles. Thus, a single street car, in effect, pre-empts as much street area, while in operation, as 6.25 private automobiles. Using

[21] "Vehicular Traffic in 1930", Detroit Rapid Transit Comm., 1930, p. 75; and "Vehicular Traffic in the Business District of Detroit", 1924, p. 18.
[22] Data furnished by E. J. McIlraith, Staff Engr., Chicago, Ill., Surface Lines.

this relation the approximate ability of a street-car lane to handle automobile traffic has been computed in Table 6 and compared with the observed data of Table 5.

TABLE 6.—ESTIMATED MAXIMUM PRACTICABLE, AND OBSERVED, USAGE OF CAR TRACKS BY AUTOMOBILES

Street cars per hour (one direction)	Estimated maximum practicable number of automobiles per hour in street-car lane	Observed number of automobiles per hour in street-car lane	Street cars per hour (one direction)	Estimated maximum practicable number of automobiles per hour in street-car lane	Observed number of automobiles per hour in street-car lane
0	750+	...	51	430	634
23	605	471	60	375	270, 347, 357, 568
27	580	385	90	190	...
30	560	275, 618	108	75	282
32	550	400	118	15	191
34	540	432, 506	120-150	0	...
36	525	265, 593	...	...	...

There are few data upon which to base information concerning the retarding effect of street railway cars on traffic in adjacent lanes. Traffic surveys in Detroit disclosed that the greatest flows (per lane) in the city were on streets having street cars.[20] These data are shown in Table 7.

TABLE 7.—MAXIMUM HOURLY MOTOR-VEHICLE TRAFFIC PER LANE, DETROIT, MICHIGAN

Location and nature of traffic	One-way roadway width, in feet	Lane No.	Hourly traffic
Grand River Avenue, at West Grand Boulevard (automobiles, street cars and buses; parking prohibited)	36	1 (curb	830
		2	890
		3 (street car)	634*
Jefferson Avenue, at East Grand Boulevard (automobiles, street cars and buses; parking prohibited)	45	1 (curb)	707
		2	895
		3	919
		4 (street car)	25†

* Also, 51 street cars; total vehicles in lane, 685.
† Private motor traffic prohibited on street-car lane, used by express street cars and local motor buses. Total vehicles in lane, 91, consisting of 41 street cars, 25 buses, and 25 private automobiles.

In view of these high records of street usage on car-line streets, which exceed those herein calculated, it seems doubtful whether any arbitrary reduction in capacity need be introduced in the present estimates because of street-car operation. If the traffic signal system is well designed, if adequate safety zones are used, if car stops are well selected, and if appropriate parking regulations are in effect, there is little reason why the lanes adjacent to the car tracks cannot carry practically the same number of vehicles per hour as similar lanes on streets without car tracks. While it is quite true that automobile speeds on street-car thoroughfares are usually lower than on automobile boulevards, because of the greater frequency of stops and the care necessarily exercised for the safety of intending passengers, the capacity of the street should not be greatly affected, as within the normal range of speeds

observed on city streets (10 to 30 miles per hr) capacities are almost constant (see Fig. 1). If, however, safety zones are not used and street cars stop very frequently the loss of movement time to passenger automobiles would seriously impair the street efficiency. If this is the case, remedial measures should be applied to improve the efficiency of motor-vehicle movement. It should be noted, however, that a street with a car track has a somewhat lower estimated lane capacity than a street used only by automobiles; for example, it will be shown later that a street with an 80-ft pavement and three moving lanes in each direction (parking allowed in curb lane), and without car tracks, has an average capacity per lane of 825 vehicles in each of three lanes; that is, one "free" lane, 900; one right-turn lane, 810; and one left-turn lane, 765; total, 2 475 automobiles. (See Table 9, Items Nos. 1, 2, and 3.) If the center lane was used by street cars, the average capacity of the two remaining lanes would be 785 vehicles per hr; one right-turn lane, 810; and one left-turn lane, 765; total, 1 575. (See Table 9, Items Nos. 2 and 3.) This is an approximate reduction of 5% in the average capacity of the free lanes, due to street railway operation.

From observations in New York City, on 42d Street between Fifth and Sixth Avenues, or 14th Street between Third Avenue and Irving Place, and on Broadway between 49th and 50th Streets, the Regional Plan of New York and Its Environs[23] prepared estimates of the total reduction in the motor-vehicle capacity of these streets because of the operation of street cars. The center lane was occupied by car tracks, the next by motor vehicles, and the third by parked automobiles. The estimates are shown in Table 8 compared to the estimated reduction obtained by the application of the preceding assumptions and the use of Table 9. It will be seen that the differences are not great, and that the capacity reductions assumed in this paper are quite liberal.

TABLE 8.—ESTIMATED REDUCTION IN THE CAPACITY OF A STREET TO HANDLE MOTOR TRAFFIC DUE TO STREET RAILWAY OPERATION, NEW YORK CITY.

Location	Number of street cars per hour (one direction)	Regional Plan estimated reduction in motor-vehicle capacity, percentages [24]	Estimated reduction in motor-vehicle capacity by application of preceding assumptions, percentages
42d Street	62	26.3	33.9
14th Street	62	25.6	34.3
Broadway	33	20.9	24.8

Effect of Motor Buses on Automobile Capacity.—To determine the effect of motor buses on other vehicle traffic is a virtual impossibility with the data available. Factors affecting bus operation are more complex than those affecting rail vehicles, which are fixed in direction and location on the street surface. It is obvious, however, that just as each street car requires an area sufficient to accommodate several automobiles, so does a bus because of its greater size and frequency of stops. Surveys on Fifth Avenue in New York City in 1926 indicated that the operation of 100 buses per hr reduced

[23] "Highway Traffic", by Harold M. Lewis and E. P. Goodrich, Members, Am. Soc. C. E., Regional Plan of New York and Its Environs, 1927, p. 114.

the total capacity of the street by about 300 automobiles. However, the crowding on Fifth Avenue is such as to hinder bus operation greatly. For satisfactory movement, it is probable that 50% more room should be allowed, making each bus equivalent, in space occupied, to 4.5 passenger automobiles.

Capacity of a Lane of Street Cars

Under favorable conditions a single electric railway track on an ordinary city street has a capacity in excess of 180 street cars per hr. However, to assure satisfactory speed, it will be assumed that the capacity is 150 street cars per hr, seating 60 passengers each, such as are used in city service on heavy-traffic thoroughfares.

Capacity of a Lane of Motor Buses

It is as difficult to determine the capacity of a lane of motor buses as it is to analyze their effect on other vehicle traffic. The reasons are practically the same in both cases.

Even were maximum bus capacities per lane possible of derivation, their value would be only academic because there are many considerations that would normally influence a decision to operate street cars or trolley buses in preference to motor buses when the density of passenger traffic is considerably below that which would require the maximum use of buses. Fifth Avenue, in New York City, is the notable exception. On this street more than 200 double-deck buses (seating, on the average, 56 passengers each) are operated southbound between 8:00 and 9:00 A. M. and approximately 190 per hr northbound between 5:00 and 7.00 P. M.

TABLE 9.—General Estimates of Surface Street Vehicle Capacities per Lane per Hour, for Various Conditions

Item No.	Type of lane	Conditions of use	Number of Motor Vehicles per Lane, per Hour	
			Developed by writer's assumptions	Report by Regional Plan of New York and Its Environs
1	Free lane	No turns	900	800*
2	Lane next to parked lane	Right turns	810	
3	Lane next to parked lane	Right and left turns	675	
4	Center lane	Left turns	765	700–750*
5	Curb lane	Right turns, no parking	450	
6	Curb lane	Right and left turns, no parking	360	
7	Car-track lane	30 street cars per hour	560	
8	Car-track lane	60 street cars per hour	375	475–600*
9	Car-track lane	90 street cars per hour	190	
10	Car-track lane	120 and more street cars per hour	0	
11	Motor-bus lane	30 buses per hour	675	.....
12	Motor-bus lane	60 buses per hour	540	.....
13	Motor-bus lane	90 buses per hour	405	.....
14	Motor-bus lane	120 buses per hour	270	.....
15	Motor-bus lane	150 buses per hour	135	.....
16	Motor-bus lane	180 buses per hour	0	.....

* General average of all lanes without reference to parking regulations.

On account of the lack of necessary data and the fact that bus operation of the character just mentioned is met with so infrequently, no bus-lane capacity will be estimated. In the later calculations of vehicle and passenger capacities of streets, however, streets with bus loadings ranging from 30 to

180 buses per hr in one direction are shown. The buses are assumed to be single-deck (suitable, for frequent stop service), of a maximum physical size considered practicable for city traffic conditions, namely, seating 40 passengers.

In Table 9, the estimated surface street capacities per lane, developed from the foregoing assumptions, are compared, as far as practicable, to those reported by Messrs. Lewis and Goodrich.[23] These values have been used in the preparation of the hourly street capacities for various widths of streets under differing conditions of use, as shown in Table 10.

TABLE 10.—AUTOMOBILE CAPACITIES OF VARIOUS STREETS UNDER DIFFERING CONDITIONS OF USE

Item No.	Descriptions	Capacity, in Motor Vehicles per Hour for One-Way Traffic (Half Street)							
		One lane		Two lanes		Three lanes		Four lanes	
		Parking prohibited	Parking permitted	Parking prohibited	Parking permitted	Parking prohibited	Parking permitted	Parking prohibited	Parking permitted
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Express highways, no grade crossings	1 500	.....	2 670	.....	3 510	.....	3 900	.....
2	Surface streets, without car tracks or buses	.....	.....	1 215	675	2 115	1 575	3 015	2 475
	Surface Streets, with Car Tracks:								
3	30 street cars per hour	.....	.....	940	420	1 775	1 235	2 670	2 135
4	60 street cars per hour	.....	.....	755	280	1 590	1 050	2 490	1 950
5	90 street cars per hour	.....	.....	570	140	1 405	865	2 305	1 765
6	120 or more street cars per hour	.....	.....	380	0	1 215	675	2 115	1 575
	Surface Streets, with Buses:								
7	30 buses per hour	.....	.....	1 025	500	1 890	1 365	2 790	2 265
8	60 buses per hour	.....	.....	910	400	1 755	1 245	2 655	2 145
9	90 buses per hour	.....	.....	795	300	1 620	1 125	2 520	2 025
10	120 buses per hour	.....	.....	680	200	1 485	1 005	2 385	1 905
11	150 buses per hour	.....	.....	565	100	1 350	885	2 250	1 785
12	180 buses per hour	.....	.....	380	0	1 215	720	2 115	1 620

The data in Table 10 are not extended beyond four lanes in each direction, nor is it believed that the assumptions herein made are applicable to streets of greater width. The table shows increases in average lane capacities as the width of the pavement increases because of the partial segregation of traffic with various objectives. On streets of greater width, it is generally believed that the effects of weaving and straddling become pronounced, with resultant decreases in the lane efficiencies. There is no authority on which to base conclusions as to the actual effect of these factors, but since urban streets with pavements accommodating more than eight lanes of traffic are seldom encountered, further consideration of this matter seems unwarranted.

STREET AREA REQUIRED PER PASSENGER IN VARIOUS VEHICLES

One of the measures of the efficiency with which a street is used by different vehicles is the street area required for each passenger transported. Usually, this is computed by dividing the static area of the vehicle by the number of passengers. A more informative method is to determine the space required for each passenger while the vehicle is in operation. This latter value may be called the operating area per passenger, and the area occupied by the moving vehicle, the operating area per vehicle.

Street Area Occupied by Various Vehicles and Their Passengers While in Motion at Approximate Capacity Conditions

The area required for the operation of an automobile may be computed from the formula:

$$A = 5\,280\,\frac{VL}{N} \qquad (2)$$

in which, A is the operating area per vehicle, in square feet; V, the average speed, in miles per hour; L, the width of lane, in feet, assumed as 10 ft; and N, the vehicles per lane per hour.

The capacity of vehicle lanes available for movement on streets with 60 to 80-ft pavements (parking permitted) is approximately 800 vehicles per hr (see Table 10, Item No. 2). If the average speed, including stops, is 15 miles per hr, the operating area per vehicle is: $A = \frac{5\,280 \times 15 \times 10}{800} = 990$ sq ft. These computations have been based on an average over-all speed of 15 miles per hr, since it is improbable that vehicle speeds on streets with heavy traffic can be maintained at much above this rate for any sustained period of time, even if the maximum speed attained is two to three times as high. For example, surveys by the Detroit Rapid Transit Commission[24] gave average motor vehicle speeds on three heavy traffic arteries, as shown in Table 11.

TABLE 11.—Average and Maximum Motor Vehicle Speeds, Detroit, Michigan

Street	Distance, in miles	Time required, in minutes	Speed, in miles per hour, average, including effect of delays	Number of stops	Maximum speed between stops, in miles per hour
Woodward Avenue	5.0	25	12.0	27	27
Gratiot Avenue	5.0	29	10.3	34	30
Grand River Avenue	5.0	28	10.7	33	25
Woodward Avenue	12.7	46	16.5	35	43
Gratiot Avenue	12.5	47	15.9	39	35
Grand River Avenue	12.6	50	15.0	41	35

The operating area of street cars and motor buses is obtained by multiplying the area used by one passenger automobile by the number of automobiles that the street car or motor bus displaces. This is, in the case of the street car, 6 190 sq ft, and, in the case of the bus, 4 455 sq ft, since the street car and the bus have been calculated to occupy space that would accommodate 6.25 and 4.50 automobiles each, respectively.

The operating area per passenger may be obtained by dividing the vehicle area by the number of passengers in each vehicle. In these calculations use has been made, in the case of the automobile: First, of the average number of passengers observed in automobiles during the rush hour in several large cities[25]; and, second, of a full seated load in a five-passenger automobile. For

[24] "Vehicular Traffic in 1930", Detroit Rapid Transit Comm., 1930, pp. 26–28.

[25] Based on surveys in Chicago, Ill., in 1929; Detroit, Mich., in 1930; Boston, Mass., in 1927; San Francisco, Calif., in 1926; St. Louis Mo., 1930; Baltimore, Md., in 1929; and Kansas City, Mo., 1930.

the motor bus and the street car the writer has used the full seated load and also the seating capacity plus a reasonable standing load. The total passenger capacity of the street car is considered to be 90 — 60 seated and 30 standing. The motor bus is assumed to have a total capacity of 50 passengers — 40 seated and 10 standing. These standards of loading are not excessive and permit reasonably comfortable movement of passengers within the car or bus. A higher proportionate standing load is used in the street car, which because of its construction and greater width, handles standees more easily than the bus.

While it is true that the standards of loading used in the case of the individual vehicle and the mass transportation vehicle are not strictly comparable, it is a matter of common observation that the automobile is seldom loaded to its full capacity, even during the rush hour. (Only about 3 of every 100 five-passenger cars observed in traffic surveys have a capacity load.) The street car and the motor bus on the other hand, not only can be loaded to maximum capacity, but actually are so loaded ordinarily during the rush period. The street car is a service vehicle catering to large numbers of persons with many different points of origin and destination, but all along some particular route. Holding itself out for hire to all who happen to be along its route, it can serve its full seated and standing capacity just as satisfactorily as it can serve a single person.

This is not true of the automobile which is essentially a vehicle for individual service, serving most satisfactorily only the personal needs of the operator (ignoring chauffeur-driven cars). It is because of this that the average number of passengers observed in each motor vehicle is rather low. Since this figure does not vary much from city to city it seems to indicate quite definitely that, regardless of the seating capacity of the private vehicle,

TABLE 12.—Operating Areas per Vehicle and per Passenger

Vehicle	Conditions	Passengers per vehicle	Operating Area, in Square Feet	
			Vehicle	Passenger
Automobile	Average rush-hour loading	1.75	990	566
Automobile	Full load	5	990	198
Motor bus	Full seated load	40	4 455	111
Motor bus	Full seated load plus 25% standing load	50	4 455	89
Street car	Full seated load	60	6 190	103
Street car	Full seated load plus 50% standing load	90	6 190	69

its practicable average rush-hour capacity does not exceed 1.75 passengers. The practicable capacity of street car and bus during rush hours, on the other hand, is approximately equal to 90 and 50 passengers, respectively. These calculations are shown in Table 12.

Passenger Capacity of a Typical Street

A more practical and useful method of comparing the efficiency of various vehicles is to apply the previously derived capacity data to a typical street and determine its vehicle and passenger-carrying potentiality when used by individual vehicles alone and when used by individual and public vehicles.

By applying the preceding data to a street of any assumed width, its capacity—first in vehicles and then in passengers—is readily ascertainable. For the purpose of demonstration the writer has selected a street with a 60-ft

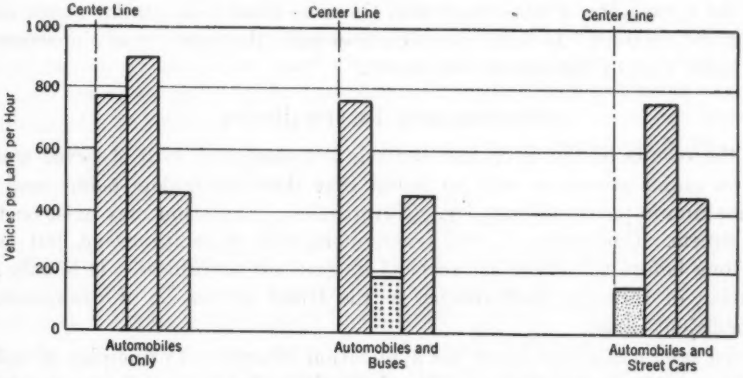

FIG. 2.—HOURLY VEHICLE CAPACITIES OF A STREET WITH 60-FOOT PAVEMENT. THREE LANES ON HALF-STREET UNDER VARIOUS CONDITIONS OF USE (PARKING PROHIBITED).

pavement on which parking is prohibited and has assumed that it is used to its maximum potential ability by: (1) Automobiles only; (2) automobiles and motor buses; and (3) automobiles and street cars. The data are compared graphically in Figs. 2 and 3.

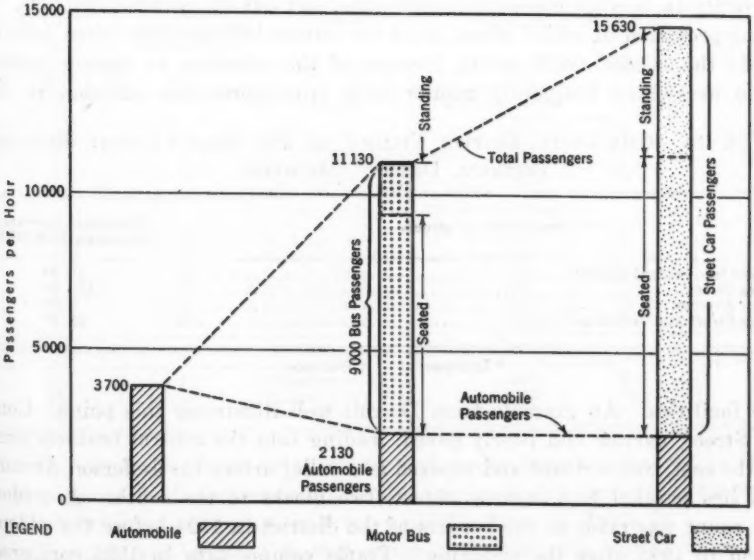

FIG. 3.—HOURLY PASSENGER CAPACITIES OF A STREET WITH 60-FOOT PAVEMENT UNDER VARIOUS CONDITIONS OF USE (PARKING PROHIBITED).

In Fig. 3, the number of passengers is based on automobiles carrying 1.75 passengers, motor buses carrying 50 passengers, and street cars carrying 90 passengers each. These are considered to be practicable average rush-hour loads. Thus, a street of average width, used both by street cars and automobiles, has a capacity of passengers 4.25 times as great as a similar street used only by automobiles. If buses replace street cars, the capacity of the street is three times that of the automobile street.

Measures for Traffic Relief

If the results of the foregoing analysis are compared with observed traffic flows on major streets, it will be found that they are seldom being used to their maximum potential capacity. Nevertheless, congestion may exist on the same streets. Congestion is not synonymous with capacity usage, but may occur long before the street has reached its greatest usefulness. It is only an indication of inherent inefficiencies in the street system or of disorganized and inefficient usage.

When such conditions occur the application of accepted principles of traffic planning may be expected to provide substantial relief at a minimum expenditure of money. It is improbable, however, that these activities alone will always be able to provide the requisite facility of traffic flow in every part of the city. Undoubtedly, a time will come when new street facilities will be considered. Ordinarily, new and widened streets are offered as the solution. Seldom, except in the very largest cities, is much attention paid to the improvement in general traffic conditions which would follow measures devised to promote the use of public transportation systems, with their acknowledged superiority in moving masses of people into, and out of, crowded districts.

The provision of added street areas for automobile use only often fails to provide the needed traffic relief because of the stimulus to greater motor-vehicle use which frequently results from any appreciable addition to the

TABLE 13.—Change in Traffic Voiume to and from Central Business District, Detroit, Michigan

Description of streets	Percentage increase or decrease, 1924-1925
All streets into central district	10.9*
Congress Street	570.0*
Jefferson Avenue	3.0†
Congress Street and Jefferson Avenue	38.1*

* Increase. † Decrease.

traffic facilities. An example from Detroit well illustrates this point. Congress Street, narrow and poorly paved, leading into the central business area from the east, was widened and repaved as a relief artery for Jefferson Avenue which lies parallel to Congress Street, two blocks to the south. A cordon traffic count was made at the borders of the district in 1924 before the widening and in 1925 after the widening. Traffic volume data in 1925 compared to 1924 are given in Table 13.

The traffic decrease on Jefferson Avenue, (Table 13), is so small that it is extremely doubtful whether any material improvement in the freedom of traffic movement resulted. The high percentage increase in traffic on the combination of Jefferson Avenue and Congress Street is entirely out of line with increases in the other parts of the area, and evidences the fact that the improved street encouraged a much greater use of motor vehicles, with a continuation of about the same degree of street crowding as existed before. It does not necessarily follow that improvements of this character are wholly undesirable; in areas of limited size and excessive street crowding, reasonable increases in the street area compatible with the normal increase in the population served, are frequently quite necessary.

Conclusion

Personal transportation on city streets is furnished in private vehicles (automobiles) and public vehicles (rapid transit trains, street cars, buses, and taxicabs). All have their place in the traffic pattern. None can be excluded and no one can be sole agency for passenger movement. All must be coordinated in any comprehensive traffic plan so that each may play its appropriate and most efficient part because, despite the enormous increase in automobile ownership, collective travel continues to be a predominant factor in urban transportation.

TABLE 14.—Modes of Transportation to and from Central Business Districts

Item No.	Order in size of cities in United States (1930 census)	City	Year of survey	Percentage of Passengers Entering and Leaving by:	
				Mass transportation vehicles	Passenger automobiles
(1)	(2)	(3)	(4)	(5)	(6)
		(*a*) Population, 1 000 000, or More			
1	2	Chicago, Ill.	1929	78.1	21.9
2	3	Philadelphia, Pa.	1928	79.2	20.8
3	4	Detroit, Mich.	1930	66.2	33.8
4	5	Los Angeles, Calif.	1924	65.3	34.7
		(*b*) Population, 500 000 to 1 000 000			
5	7	St. Louis, Mo.	1930	65.4	34.6
6	8	Baltimore, Md.	1929	60.8	39.2
7	9	Boston, Mass.	1932	65.7	34.3
8	10	Pittsburgh, Pa.	1927	74.6	25.4
9	11	San Francisco, Calif.	1926	74.5	25.5
10	12	Milwaukee, Wis.	1926	50.4	49.6
		(*c*) Population, 250 000 to 500 000			
11	14	Washington, D. C.	1930	36.7	63.3
12	19	Kansas City, Mo.	1930	48.9	51.1
13	24	Louisville, Ky.	1926	53.6	46.4

The service rendered by public transportation is best measured, perhaps, by the proportion of workers and shoppers transported to and from central business districts, as summarized from available data, in Table 14.

Taxicabs are included in the values in Column (6), except in the case of Louisville (Item No. 13) where they are included among the mass trans-

portation vehicles (Column (5)). Item No. 1, of Table 14, covers twelve hours of the business day. Of the automobiles crossing the boundary line of the central business district, more than 30% passed through without stopping for the transaction of any business. In rush hours, the percentage of through vehicles remains substantially the same, although passengers outbound in automobiles in the rush hour comprise only 11% of the total outbound passenger movement. Further analysis shows that of the total people in the central business district at any one time, never more than 7% had arrived by automobile; the others had come by means of public transportation systems.

If it is assumed that the cities in Table 14 are typical of others in their respective population classes, approximately two-thirds of all the 29 000 000 people in the cities of more than 250 000 population, are dependent on public transportation for their daily travel to and from business areas. It is noted that the proportion that uses the public carrier generally increases with the size of the city, and since the trend in National growth, at least up to 1930, has been toward the city, it is quite likely that public transportation because of its high passenger-carrying ability, may assume a greater importance than in the past. Briefly, the facts surrounding urban and rural growth are:

1.—Approximately one-fourth of all the people of the United States in 1930 lived in the 37 cities of 250 000 population and more. In 1890, less than one-eighth lived in the 11 cities in this class.

2.—One-third of the population lived in the remaining 3 128 cities of 2 500 to 250 000 population. In 1890, it was less than one-quarter.

3.—In 1930, 43.8% of the population lived in rural territory. In 1890, 64.6% of the total lived outside of urban areas.

4.—Urban population has more than tripled in the last 40 years; rural population has increased a bare one-third.

No growing city can cope successfully with the traffic problem of the present or prepare for the more complex conditions of the future unless it simultaneously carries on two different types of traffic activities: (*a*) Employing such means as are immediately available to obtain higher efficiency from existing services; and (*b*) constructing additional traffic facilities to provide for the future normal increase in traffic volume.

Neither of these types can be prosecuted to the total or even partial neglect of the other. In each instance, it is important that appropriate consideration be given to both public and private vehicles and the facilities for their operation. The amount of money expended in furnishing facilities for each type of vehicle should be in proportion to the economic importance of each in the city plan.

Acknowledgment

The writer is indebted to the members of the Committee on Street Traffic Economics of the American Transit Association, for valuable assistance in furnishing fundamental data bearing on street usage and for their critical comments during the preparation of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

WATER-POWER DEVELOPMENT OF THE ST. LAWRENCE RIVER[1]

BY DANIEL W. MEAD,[2] HON. M. AM. SOC. C. E.

SYNOPSIS

An analysis of the proposed official plans for the development of water power along the International Section of the St. Lawrence River, is contained in this paper. Since the Wooten-Bowden Report in 1921 on the development of the proposed St. Lawrence Waterway, there has been controversy between the United States and Canadian members of the various engineering boards that have made reports on the development of the International Section, in regard to the relative merits of a single-stage and a two-stage plan of development of the water-power resources. The American engineers have consistently advocated a single-stage plan, and the Canadian engineers have consistently advocated a two-stage plan.

In December, 1931, the Joint Board of Engineers (United States and Canada), consisting of an entirely new membership of the United States Section and of the original members of the Canadian Section, was reconvened and in March, 1932, was instructed to reach an agreement on a plan for early execution. On April 9, 1932, the Joint Board submitted the "Reconvened" Report on which the treaty now under consideration is based.

If this analysis is correct, the value of the two-stage plan will be nearly $95 000 000 less than the alternate Joint Board plan for a single-stage development at Barnhart Island, and at least $154 000 000 less than the New York single-stage plan. The advantages of the various plans are not adequately discussed in the report of the Joint Board, and no reference is made in the Joint Board Reconvened Report as to why the two-stage plan is recommended. The reasons set forth in the Conference Report (Canadian) seem entirely inadequate to offset the aforementioned disadvantages. If the reports and this analysis is correct, it will be a very serious mistake and a misfortune to both the United States and Canada, in the writer's opinion, if the treaty now pending is ratified on the basis of the Joint Board Reconvened Report.

[1] Presented at the Joint Meeting of the Power Division of the Society with the American Institute of Electrical Engineers and the Hydraulics Division of American Society of Mechanical Engineers, Chicago, Ill., June 29, 1933. Discussion on this paper will be closed in November, 1933 *Proceedings*.

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Introduction

With his limited knowledge of the International Rapids Section of the St. Lawrence River, the writer, ordinarily, would not have presumed to discuss this important question. As a disinterested engineer familiar with water-power development, he was requested by the Power Division of the Society to examine the various reports on this subject and to analyze the comparative values of the principal plans on the basis of these reports.

A proper report should be so prepared that it will give any intelligent engineer, familiar with the character of the work considered, fair understanding of the reasons for the recommendations made in the report. As an American citizen, and especially as an hydraulic engineer interested in water-power development, the writer believes that it is his right to know, in the case at hand, the reasons for the recommendations, on the basis of which an important treaty is expected to be consummated, and to judge of the validity of such recommendations from the information furnished in these reports.

If his interpretation of the conditions is faulty or if his conclusions are in error, he believes that he is entitled to have such conditions explained and such errors corrected. His mind is entirely open on the entire subject, but he believes that the people of the United States and Canada who are asked to spend hundreds of millions of dollars on this project are entitled to something better than an opinion, no matter from how distinguished a source, and to consider this project on the broader economic basis on which the writer has attempted to examine it.

The investigation leading to the treaty, now (1933) before the United States Congress, began with the work of Col. W. P. Wooten, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and Mr. W. A. Bowden, Canadian Government Engineer, in 1920–21. The results of their investigation were outlined in their report to the International Joint Commission, dated June 24, 1921.[3]

This Board was so restricted in time that it was unable to go into the subject in great detail, but submitted a set of recommendations on which its members were able to agree, and pointed out certain others on which they differed. This report was considered by the International Joint Commission in 1921, together with certain alternate suggestions, submitted by or on behalf of the Hydro-Electric Power Commission of Canada, The New York and Ontario Power Company, Hugh L. Cooper and Company, of New York City, and Mr. S. M. Lovelace, of Montreal, Que., Canada.

The Commission accepted the plans of the Engineering Board as a basis for the recommended improvements of the St. Lawrence River with the suggestion that both the plans of the Engineering Board and the alternate plans should be referred to a larger board for final determination.[4]

In 1924 a Joint Board of Engineers was appointed, consisting of six members, three from the United States and three from Canada. This Board was asked to review the Wooten-Bowden Report and to extend its inquiries to certain additional matters relevant to proposals made by the International

[3] See Senate Doc. No. 179. 67th Cong., 2d Session.
[4] See Senate Doc. No. 114, 67th Cong., 2d Session.

Joint Commission in its report dated December 19, 1921. The studies of this Joint Board extended over several years, and its report was submitted in 1926. Most of the engineering questions were cleared up by this report, and a large number of questions of fact and opinion which were referred to it, were definitely answered, but the Board failed to agree on the best method of developing power and navigation in the International Rapids Section.[5]

The Canadian Section favored a two-dam plan (see Fig. 1 (*A*)) with the upper dam at Ogden Island, and the lower near the foot of the Rapids. The American Section, however, favored a single dam (see Fig. 1(*B*)), near the foot of the Rapids. Both Sections agreed that both plans were feasible from an engineering standpoint, and suggested that the choice between them be determined by constructing agencies at a later date; they also agreed that the two-dam plan would cost about $30 000 000 more than the single-dam plan.

After several years, in which the subject was studied further, a Conference Board of Engineers was appointed in Canada. This Board recommended moving the upper dam in the two-dam plan from Ogden Island to Crysler Island, farther down the river.[6] By this change the difference in first cost between the one-stage and the two-stage plans was increased to about $40 000 000. The change in location also resulted in raising the proposed water level along the Iroquois-Morrisburg Section to such height that the problem of protecting this section, which had been in controversy before, became substantially the same in the two plans proposed.

In 1930, the State of New York established a Commission which was instructed to report on "a comprehensive plan for the development and operation of the water power resources of the State on the St. Lawrence River. * * *" This Commission reported[7] under date of January 15, 1931, and its engineers advised:

"A modification of the one-dam plan which eliminates the unusual serious hazards of the previous plans and makes possible the development of a somewhat increased amount of power. The estimated saving in first cost under this plan as compared with the two-stage Crysler Island plan was increased further to about $70 000 000.

"In December, 1931, the Joint Board of Engineers was instructed to meet and report on a mutually satisfactory plan for the improvement of the International Rapids Section.[8]

"In March, 1932, the Board was instructed to reach an agreement on a plan for early execution that would provide effectively for navigation and power requirements."[9]

It should be noted that the Canadian members of the Joint Board of Engineers were appointed in 1924 and consisted of Mr. Duncan W.

[5] See Rept. of Joint Board of Engrs., with Appendix, dated November 16, 1926, Ottawa, Ont., Canada, 1927.

[6] See Rept. of Conference of Canadian Engrs. on the International Rapids Section, with Appendix and plates, dated December 30, 1929, Ottawa, Ont., Canada, 1930.

[7] See Rept. of the St. Lawrence Power Development Comm., dated January 15, 1931, Albany, N. Y.

[8] See Rept. of Joint Board of Engrs. (Reconvened), p. 4, Paragraph 16.

[9] *Loc. cit.*, p. 4, Paragraph 7.

McLachlan, of the Department of Railways and Canals, of Ottawa, Olivier O. Lefebvre, M. Am. Soc. C. E., Chief Engineer, Quebec Streams Commission, of Montreal, and Charles H. Mitchell, M. Am. Soc. C. E., of Toronto. They have continued unchanged from the time of their appointment to the present (1933). The original United States members of the Board consisted of the late Lt.-Gen. Edgar Jadwin, U. S. A., M. Am. Soc. C. E. (then Chief of Engineers, U. S. A.), Col. William Kelly, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and Brig.-Gen. G. B. Pillsbury, U. S. A., M. Am. Soc. C. E., (then Colonel, Corps of Engineers, U. S. A.); while the United States members of the Reconvened Joint Board of 1930 consisted of Col. H. B. Ferguson, Corps of Engineers, U. S. A., *Chairman,* Col. E. M. Markham, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and General Pillsbury. In June, 1930, Maj. Eugene Reybold, Corps of Engineers, U. S. A., was detailed to replace General Pillsbury.

It is unfortunate that the same members of the Board of United States Engineers were not employed continuously on this project. New board members appointed to co-operate with others who have served for many years on a project as involved as that of the navigation and power development of the International Section of the St. Lawrence River, are at a great disadvantage. The various plans that might be adopted are so numerous and the possible method of effecting the same results are so involved that only long and continued contact with the problem will insure the familiarity necessary to secure the best results. Such conditions, together with the instructions under which the Board was working, necessitated coming to a conclusion on a "mutually satisfactory plan" which could "provide effectively for navigation and power requirements" and did not involve either the "best" or the "most economical" development for navigation or for power.

It is also unfortunate that the economic desirability of river and harbor improvements is usually determined by legislative bodies or by non-engineering fact-finding boards. The engineering boards appointed to plan such improvements are seldom expected to look into the economic phases of such problems in their broader aspects. First cost, while always of importance, must yield to various questions of safety, expedience, and influence. Apparently, for the foregoing or for similar reasons, none of the engineering reports considered, except that of the New York Board (which is confined principally to the question of water-power development), discusses the projects involved from an economic standpoint either as to navigation or water-power development, except as to the items of first cost.

As navigation works are seldom if ever constructed from the avails of bond issues, but from annual or occasional appropriations, the matter of interest during construction is almost never considered. In the case at hand, it is expected that the water-power development will be undertaken by the State of New York and the Province of Ontario, Canada, and will involve bond issues and a considerable expense of interest during construction. Other economic considerations, such as cost of maintenance, operation, depreciation, amortization, etc., also become of importance quite equal perhaps to the matter of first cost. It would seem, therefore, that those interested in this

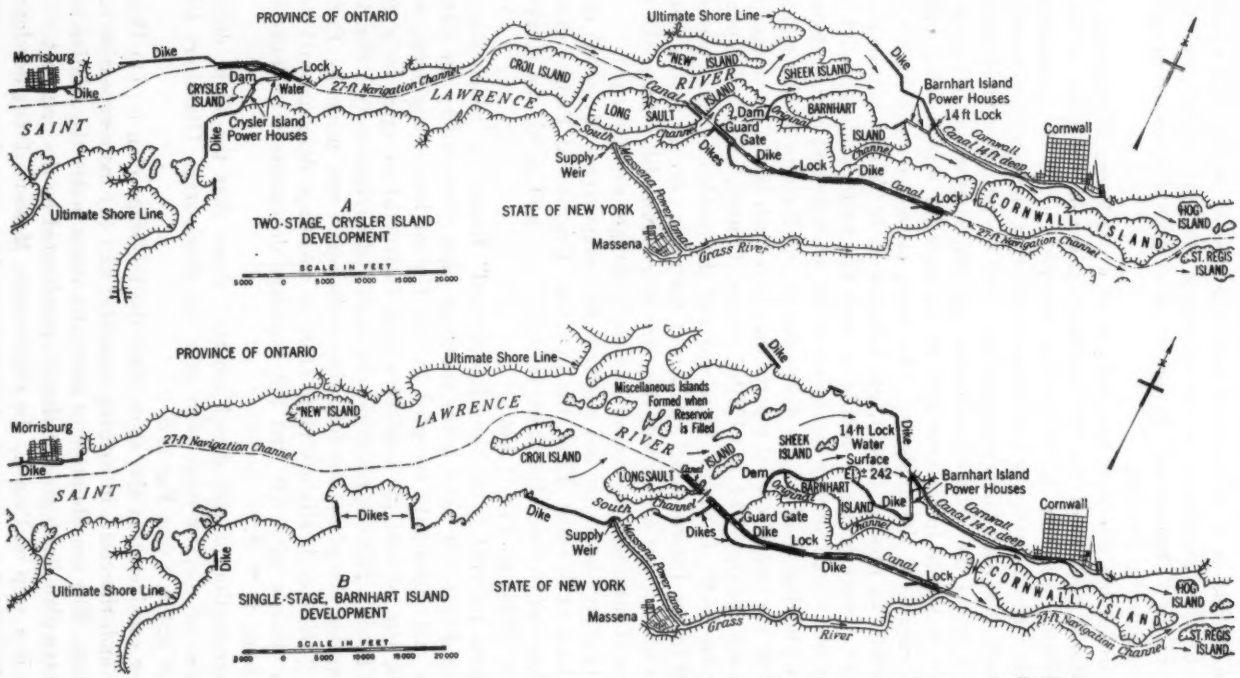

FIG. 1.—COMPARISON OF TWO-DAM AND SINGLE-DAM PLANS FOR DEVELOPING ST. LAWRENCE RIVER.

project, which involves so great an expense and which is of so great importance to thousands of citizens of both the United States and Canada, are entitled to some knowledge of the economic relations of the project recommended in the "Reconvened" Report to other plants which seem equally or more economically feasible.

The writer has analyzed the problem of water-power development on the International Section of the St. Lawrence River on the basis of the plans and estimates presented in the various reports and on the assumption that such plans are adequate for the purpose and that such estimates are reasonably correct, or at least fairly comparative. He has found certain discrepancies between some of the detailed estimates and the summaries, but such differences are of minor importance and in no material way do they affect the ultimate results of the analyses. The costs, estimated on a higher market than prevails at present (1933), are believed to be fairly comparative; any attempt to correct prices or details to meet the writer's ideas would involve confusion and would destroy the values of the analyses and comparisons made.

The writer has confined his discussion to the three most important plans proposed in the several official reports that have been issued by various boards of engineers appointed by the United States, Canada, and New York State to report on the St. Lawrence Waterway and the incidental water power to be developed in the International Section of the river. So far as the plans and estimates of cost are concerned, they are taken unmodified from the official reports or, if modified, the reasons for such changes are discussed.

In this paper the writer has analyzed the comparative advantage of three official plans, as follows: First, the two-stage Reconvened Plan; second, the single-stage Conference (Barnhart Island) Plan; and third, the single-stage New York (Massena Point) Plan.

The plan referred to in the "Reconvened" Report on which the treaty under consideration is based is the two-stage plan advocated by the Canadian engineers and apparently accepted by the United States engineers, not on its merits, but because the engineers were instructed to agree by a certain date on a feasible plan for power development and navigation (see Fig. 1(*A*)).

Concerning the plan, the following statement[10] was made by President Hoover: "The cost is slightly more (for the two-stage development) but the Canadian officials have felt that the two-stage development is desirable for many reasons * * *."

An examination of the official reports,[11] shows that the slight difference in first cost of the two-stage plan over the single-stage (Barnhart Island) plan is $38 324 000 (see Fig 1(*B*)).

Further consideration also shows that the two-stage plan is: (1) Deficient in output and, therefore, in earning capacity; and (2) more expensive in cost of operation. The consideration of these two items makes the relative cost of the two-stage plan much greater than is represented by the first cost.

Fig. 2 is a map of Plan C, the single-stage, Massena Point development, and Fig. 3 shows the profiles of all three plans, for comparison.

[10] Treaty, p. 11.
[11] Reconvened Rept., p. 9, and Conference Rept., p. 11.

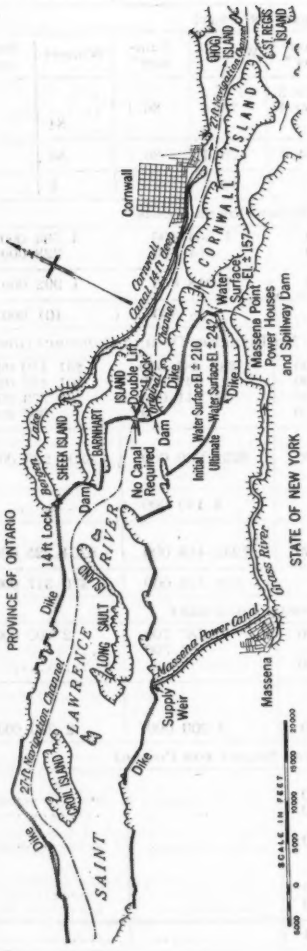

FIG. 2.—PLAN C; SINGLE-STAGE MASSENA POINT DEVELOPMENT.

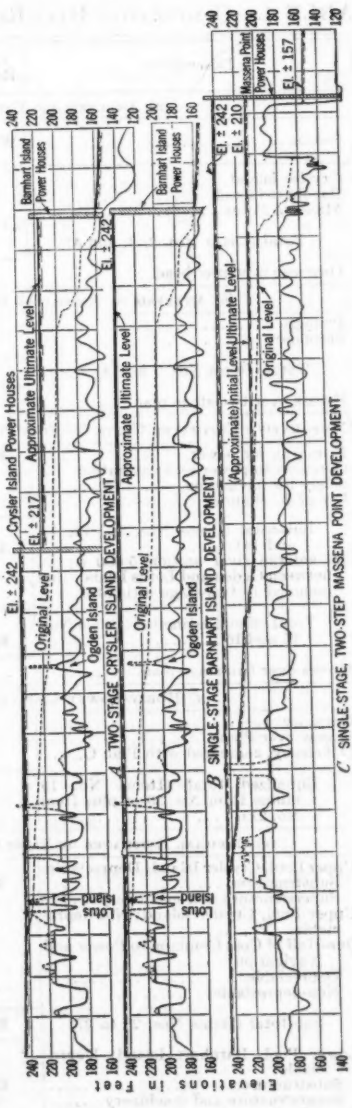

FIG. 3.—COMPARATIVE PROFILES, PLANS A, B, AND C.

CONTROLLING PHYSICAL CONDITIONS

In all computations in the various reports involving stream flow it has been assumed that the summer flow (for nine months of each year) will be 235 000 cu ft per sec, and that during the three months of winter the flow will be reduced to 210 000 cu ft per sec.

The estimated head available in the three plans considered is shown in Table 1(*a*) and the horse-power years calculated on the basis of the stream flow are given in Table 1(*b*). Table 1(*c*) shows a summary of official esti-

TABLE 1.—COMPARATIVE DATA RELATING TO THE THREE PLANS ANALYZED

Item No.	Description	(A) Two-stage, Reconvened Plan		(B) Single-stage, Conference Plan		(C) Single-stage, New York Plan	
	(*a*) ANALYSIS OF ESTIMATED HEAD (IN FEET)						
1	Season	Winter*	Summer*	Winter*	Summer*	Winter†	Summer†
2	Crysler Island	19.6	24.2	.....	.....	.....	.....
3	Barnhart Island	56.4	60.4	76.4	86.1	.....	.....
4	Massena Point	.....	.....	.....	.....	84	88
5	Total (Items Nos. 2, 3, and 4)	76.0	84.6	76.4	86.1	84	88
6	Decrease in winter head	8.6	.....	9.7	.....	4	.....
	(*b*) ANALYSIS OF ESTIMATED POWER (IN HORSE-POWER YEARS)						
7	Primary	1 596 000		1 604 400		1 764 000	
8	Secondary	294 100		314 200		228 000	
9	Total (Items Nos. 7 and 8)	1 890 100		1 918 600		1 992 000	
10	Excess over two-stage plan	.....		28 500		101 900	
	(*c*) COMPARATIVE ESTIMATED COSTS (27-FOOT NAVIGATION; NO INTEREST DURING CONSTRUCTION)						
11	Primarily for power	$132 452 000		$87 481 000		$81 150 000	
12	Common to power and navigation	108 102 000		117 047 000		101 495 000	
13	Solely for navigation	34 188 000		27 741 000		20 773 000	
14	For 27-ft. channel	0		0		807 000	
15	Sub-total (Items Nos. 11, 12, 13, and 14)	$274 742 000		$232 269 000		$204 225 000	
16	To enlarge river section 5 000 sq. ft. between Ogden and Lotus Island (not included in two-stage estimate)	0		4 149 000		.....	
17	Total official estimates (Items Nos. 15 and 16)	$274 742 000		$236 418 000		$204 225 000	
18	Excess over two-stage plan	.....		$38 324 000		$70 517 000	
	(*d*) HORSE-POWER CAPACITY OF PROPOSED EQUIPMENT						
19	Capacity	2 199 960		2 287 700		2 200 000	
20	Excess over Plan C	.....		—87 700		.....	
21	Deficiency, compared with Plan C	+40		.....		.....	
22	Equalized total (Items No. 19 minus Item No. 20 or plus Item No. 21)	2 200 000		2 200 000		2 200 000	
	(*e*) OFFICIAL ESTIMATES OF FIRST COST (WORKS SOLELY FOR POWER)						
	Upper Pool, Crysler Island; Depreciable:						
23	Substructures	$18 987 070		.....		.....	
24	Superstructure and machinery	30 611 680		.....		.....	
25	Upper Pool, Crysler Island: Not depreciable	5 906 250		.....		.....	
	One-Half of Cost Common to Power and Navigation:						
26	Depreciable	6 763 500		.....		.....	
27	Non-depreciable	31 154 500		.....		.....	
28	Sub-total (Items Nos. 23 to 27)	$93 423 000		.....		.....	
	Lower Pool, Barnhart Island; Depreciable:						
29	Substructure	$21 311 870		$37 819 000		$27 526 500	
30	Superstructure and machinery	43 249 020		49 662 000		47 179 100	
31	Lower Pool, Barnhart Island; not depreciable	12 386 070		4 149 000		6 444 400	
	One-Half of Cost Common to Power and Navigation:						
32	Depreciable	5 795 700		11 119 500		14 926 500	
33	Non-depreciable	10 337 340		47 404 000		35 821 000	
34	Sub-total (Items Nos. 29 to 33)	$93 080 000		$150 153 500		$131 897 500	
35	Official totals (Item No. 28 plus Item No. 34)	$186 503 000		$150 153 500		$131 897 500	

* Conference Rept., pp. 10 and 11.
† New York Rept., p. 45.

TABLE 1.—(*Continued*)

Item No.	Description	A) Two-stage, Reconvened Plan	(B) Single-stage, Conference Plan	(C) Single-stage, New York Plan
	(*f*) ANNUAL EXPENSE			
36	Five per cent. interest on investment (5% of Item No. 35)	$9 325 150	$7 507 675	$6 594 875
37	Salaries	633 000	367 000	367 000
38	Station material and supplies	252 000	144 000	144 000
39	Maintenance and repairs to structures (0.125%)	158 398	123 251	112 040
40	Liability insurance (11.5% of annual payroll)	72 795	42 205	42 205
41	Management	500 000	500 000	500 000
42	Depreciation (1.75% of depreciable property)	2 217 580	1 725 509	1 568 562
43	Amortization (1% of capital cost, Item No. 35)	1 865 030	1 501 535	1 318 975
44	Total annual expense (Items Nos. Nos. 36 to 43)	$15 023 953	$11 911 175	$10 647 657
	(*g*) COMPARISON OF POWER PLANT COSTS BASED ON OFFICIAL ESTIMATES (ITEM NO. 35)			
45	Estimated total power (horse-power years; see Item No. 9)	1 890 100	1 918 600	1 992 000
46	Excess over two-stage plan	.....	28 500	101 900
47	Estimated capital cost per primary horse-power year $\left\{\frac{\text{Item No. 35}}{\text{Item No. 7}}\right\}$	$116.96	$93.59	$74.77
48	Estimated capital cost per total horse-power year $\left\{\frac{\text{Item No. 35}}{\text{Item No. 9}}\right\}$	98.67	78.26	66.21
49	Difference in favor of Plans B and C (capital cost per total horse-power year)	.....	20.41	32.46
50	Total income with primary power at $15 and secondary power at $10	26 881 000	27 208 000	28 740 000
51	Total annual expense (Item No. 44)	15 023 953	11 911 175	10 647 657
52	Net income	$11 857 047	$15 296 825	$18 032 343
53	Excess annual income of Plans B and C over Plan A	.....	3 439 778	6 235 296
54	Difference, capitalized at 5%	.....	68 795 560	124 705 920
	(*h*) PARTLY CORRECTED CAPITAL COST			
55	Official estimate of capital cost (Item No. 35)	$186 503 000	$150 153 500	$131 897 500
56	Additional cost of 5 000 sq. ft. of excavation	4 149 000	.....	.....
57	Additional 40 hp. deficient capacity	1 600	.....	.....
58	Deduct 87 700 hp. excess capacity	.....	3 508 000	.....
59	Corrected estimate of capital cost	$190 653 600	$146 645 500	$131 897 500
	(*i*) ANNUAL EXPENSE BASED ON PARTLY CORRECTED CAPITAL COST			
60	Five per cent. interest on investment (Item No. 59)	$9 532 680	$7 332 275	$10 647 657‡
61	Salaries	633 000	367 000	.....
62	Materials and supplies	252 000	144 000	.....
63	Maintenance and repairs (structures, 0.125%)	158 400	118 866	§
64	Liability insurance (11.5% of annual payroll)	72 795	42 205	.....
65	Management	500 000	500 000	.....
66	Depreciation (1.75%)	2 217 608	1 664 119	.....
67	Amortization (1% of capital cost)	1 906 536	1 466 455	.....
68	Total annual expense	$15 273 019	$11 634 920	$10 647 657

‡ See Item No. 44.
§ No correction.

TABLE 1.—(*Continued*)

Item No.	Description	(A) Two-stage, Reconvened Plan	(B) Single-stage, Conference Plan	(C) Single-stage, New York Plan
	(*j*) Comparison of Power Plant Cost Based on Partly Corrected Estimate (Item No. 59)			
69	Corrected installed capacity (Item No. 22)	$2 200 000	$2 200 000	$2 200 000
70	Total estimated power (horse-power years (Item No. 9)	1 890 100	1 918 600	1 992 000
71	Excess power (horse-power years) Plans B and C over Plan A	.....	28 500	101 900
72	Capital cost per horse-power year { Item No. 59 / Item No. 9 }	100.87	76.43	66.21
73	Difference in favor of Plans B and C	.....	24.44	34.66
74	Net annual income (Item No. 50 minus Item No. 68)	11 608 000	15 573 080	18 092 343
75	Excess net income, Plans B and C over Plan A	.....	3 965 080	6 484 343
76	Excess income (Item No. 75) capitalized at 5%	.....	79 301 600	129 686 860
	(*k*) Additional Corrections of Official Estimates (for Head)			
77	Increased head, in feet	—1.3 ‖	+1.5 ¶	.....
78	Estimated change in primary power (horse-power years)	—27 300	+31 500	.....
79	Estimated change in secondary power (horse-power years)	—2 400	—23 600	.....
80	Estimated change in total power (horse-power years)	—29 700	+7 900	.....
81	Change in annual income	—$433 500	+$236 500	.....
	(*l*) Comparison of Plans Based on Corrected Estimates and Heads (See Item No. 59)			
82	Corrected installed capacity (Item No. 22)	2 200 000	2 200 000	2 200 000
83	Primary horse-power years	1 568 700	1 635 900	1 764 000
84	Secondary horse-power years	291 700	290 600	228 000
85	Corrected total horse-power years (Item No. 70 minus Item No. 80)	1 860 400	1 926 500	1 992 000
86	Excess horse-power years, Plans B and C over Plan A	.....	66 100	131 600
87	Estimated cost per primary horse-power year { Item No. 59 / Item No. 83 }	$121.54	$89.64	$74.77
88	Estimated cost per total horse-power year { Item No. 59 / Item No. 85 }	102.48	76.12	66.21
89	Difference in favor of Plans B and C in cost per total horse-power year	.....	26.36	36.27
90	Corrected total income with primary power at $15 and secondary power at $10	26 447 500**	27 444 500††	28 740 000
91	Correct total annual expense (Item No. 68)	15 273 019	11 634 920	10 647 657
92	Net income (Items Nos. 90 minus Item No. 91)	$11 174 481	$15 809 580	$18 092 343
93	Difference in net income	.....	$4 635 099	$6 917 862
94	Difference capitalized at 5%	.....	92 701 080	138 357 240
	(*m*) First Cost with Respect to Both Power and Navigation (27-Foot Navigation; No Interest Charges)			
95	Works for power, plus one-half work for navigation and power (Item No. 35)	$186 503 000	$150 153 500	$131 897 500
96	Excess of Plan A over Plans B and C	.....	(36 349 500)	(54 605 500)
97	Works for navigation plus one-half work for navigation and power	88 239 000	86 264 500	72 327 500
98	Excess of Plan A over Plans B and C	.....	(1 974 500)	(15 911 500)
99	Total first cost (Items No. 95 plus Item No. 97)	$274 742 000	$236 418 000	$204 225 000
100	Total excess of Plan A over Plan B and Plan C	.....	($38 324 000)	($70 517 000)

‖ Twelve months.
¶ Three winter months.
** Reduced for loss by power; see Table 1 (*k*).
†† Increased for extra winter power; see Table 1 (*k*).

mates of the total cost of the three combined navigation and water power projects.[12]

In neither the Reconvened Report nor in the Conference Report does the writer find any estimates of operating costs. Therefore, he has based his estimates (see Table 2) on the New York Report, Appendix A, and while he would personally change these estimates in some respects, he believes that the official figures used are comparatively correct and suitable for this analysis.

TABLE 2.—Operating Charges (One Plant, Canada or United States) from New York Report, Appendix A.

Salaries	Single-stage		Extra for two-stage		Total two-stage
Direct Operation Staff:					
Chief engineer (1)	$12 000		.....		.....
Electrical engineer (1)	8 000		.....		.....
Hydraulic engineer (1)	8 000		.....		.....
		$28 000	.....		.....
Draftsmen (4)	$12 000		(2) $6 000		.....
Accountant (1)	4 000		(1) 4 000		.....
Clerks (4)	8 000		(2) 4 000		.....
Telephone operators (3)	3 000		(3) 3 000		.....
		27 000		$17 000	.....
		$55 000			
Power-House Operation:					
Station superintendent (1)	$6 000		$6 000		.....
Assistant superintendents (3)	12 000		12 000		.....
Dispatchers (3)	11 000		11 000		.....
Group chiefs (7)	17 500		(6) 15 000		.....
Operators (14)	28 000		(12) 24 000		.....
		$74 500		$68 000	.....
Outside Operation:					
Regular and extra help on dam	$30 000		$25 000		.....
Sub-station	6 000		6 000		.....
Oil and waste	4 000		3 000		.....
		$40 000		$34 000	.....
Maintenance and Repairs:					
Chief electrician (1)	$3 000		$3 000		.....
Master mechanic (1)	3 000		3 000		.....
Mechanics (4)	8 000		8 000		.....
		$14 000		$14 000	.....
Total payroll (one plant)		$183 500		$133 000	.....
Total payroll two-stage (four plants): 2 ($183 500+133 000)	.....		.....		$633 000
Materials and Supplies:					
Regular and extraordinary at $3 000 per unit		$72 000		$54 000	.....
Regular and extraordinary at $3 000 per unit for two-stage (four plants) 2 ($72 000+$54 000)	.....		.....		$252 000
Total for single plant		$255 500		$187 000	.....
Total for two-plant		$511 000		$374 000	.....
Total for four-plant (two-stage) ($511 000+$374 000)	.....		.....		$885 000
Estimated annual operating cost	New York Plan (Single-Stage)				$511 000
	Conference Plan (Single-Stage)				511 000
	Reconvened Plan (Two-Stage)				885 000

First Analysis

In the first analysis the writer used the official figures as given in the official reports, without exception, taking operating charges from the New York Report. The cost figures for each water-power project are made up of:

[12] Reconvened Rept., p. 9; Conference Rept., p. 11; and, New York Rept., p. 50, respectively.

(1) The official estimate of the cost of works to be used solely for power; and (2) one-half the official estimate of the cost of works common to navigation and power.

While it is possible that some other proportion of this cost of the work common to navigation and power may be charged to the cost of power, such change in cost will apply to all plans and will not seriously affect the comparisons made.

In Table 1(*e*) are given the official estimates of the cost of the plans subdivided into depreciable and non-depreciable elements for the purpose of calculating the annual expense for each plan. In Table 1(*f*) are given estimates of the annual expense, of each of the three plans, made on the basis used in the New York Report, except that annual depreciation is taken at $1\frac{3}{4}\%$ of the value of depreciable property and annual amortization is calculated on the basis of 1% of the total capital cost.

In Table 1(*g*) are given; (*a*) The capital cost (Item No. 35), installed capacity (Item No. 19), and the estimated horse-power years output (Item No. 45); (*b*) the capital cost per horse-power year (Item No. 48); (*c*) the annual earnings based on $15 per h-p year for firm power and $10 per h-p year for secondary power (Item No. 50); (*d*) the excess capital costs per horse-power year of the two-stage plan over the single-stage plans (Item No. 49); (*e*) the earning capacity of each plant (Item No. 52); (*f*) the excess earning capacity of the single-stage plans over the two-stage plan (Item No. 53); and (*g*) the capitalized value of the excess earning capacities (Item No. 54).

Based on the unchanged estimates of the official reports, Table 1(*g*) shows that when the full financial conditions of power development are considered the relative cost of the two-stage plan is: (*a*) Almost $69 000 000 more than the single-stage Conference Plan; and (*b*) almost $125 000 000 more than the New York Plan.

Second Analysis

In the first analysis official estimates have been used so as to make the comparisons as far as possible independent of the writer's interpretations of these reports.

There are, however, certain estimates in these reports which the writer believes to be clearly in error, and which should be corrected to make the plans truly comparative:

(*a*) In the Conference Plan, $4 149 000 of capital cost is included to secure 5 000 sq ft more cross-section in the area between Lotus Island and Morrisburg.[13]

The writer understands that the Canadian engineers consider 95 000 sq ft as entirely adequate in this section for the two-stage plan, but that the single-stage plan will require a further increased section due to the manner of its operation, its pool fluctuations in winter, and certain ice conditions which might change the gradient so as to reduce the section between Ogden

[13] See Conference Rept., p. 5, Paragraph 16 (*b*).

and Lotus Islands sufficiently to increase velocities above the limit necessary to permit the formation of surface ice and prevent the formation of frazil ice.

With his limited knowledge of the local conditions, the writer is unable to concede the probability of any such occurrence. In the two-stage plan the fall to Crysler Island must be equal to the head necessary to produce the flow through the section.

At Crysler Island a certain head is utilized, and a head of from 1.3 to 1.5 ft will be lost in passing the water through the power-house and turbines. In the same way a head sufficient to cause the flow through the lower section will be required from the Crysler Island tail-water to the power-house at Barnhart Island.

If the Crysler Island power-house is eliminated there will be an increase of 25 ft in depth for the 18-mile reach from the Barnhart Island plant to Crysler Island, thus reducing the head necessary to create the flow in the lower reach of the river, and eliminating also the head lost in the two-stage power-house at that point. Certainly with any reasonable operation the tendency should be to increase rather than to decrease the depth of water in the reaches above Crysler Island. Truly, ice troubles may follow mismanagement with either plan, but would seem more likely to occur with the two-stage plan and its exceedingly limited pondage. With proper design and proper operation, trouble with frazil ice will be confined to the early period of winter just before surface ice is formed, and should be far less with the single-stage plan at or near Barnhart Island with its extra depth and consequent lower velocities. If there is any valid reason for including this extra expense in the single-stage and not in the two-stage development, it is not obvious. This amount, therefore, is added to the cost of the two-stage development.

(*b*) In the estimate of installed horse-power of machines, the single-stage Conference Plan is penalized by adding 87 740 hp in machine equipment above that used in the two-stage plan, and 87 700 hp more than is provided in the New York Plan.

This extra capacity may be desirable for the equipment of the plant in question but if so there would be a necessity for increased capacity in the two-stage and New York plans. For the purpose of comparison, the equipment of the plants is equalized on the basis of a total capacity of 2 200 000 hp.

The estimated cost of installation per horse-power is shown in Table I of the Joint Report (page 281) and is segregated as follows:

Item 1 (*b*) Power-house substructure.....	$24 000 000
Item 1 (*e*) Tail-race excavation.....	3 799 000
Item 1 (*g*) Superstructure and machinery....	53 571 000
Total	$81 370 000
Overhead, 12.5% (see Joint Report, page 287).	10 171 000
Total cost of 2 287 000 installed horse-power..	$91 541 000
Approximate cost per horse-power.....	$40

The cost of the 40 hp additional equipment, which is recognized as quite insignificant, is added therefore to the two-stage plan, and the cost of 87 700 hp extra equipment (at $40 per hp) is deducted from the cost of the single-stage Conference Plan to make the plans fairly comparative The foregoing corrections to the official estimates is shown in Table 1(*h*).

The annual expense involved is calculated in Table 1(*i*) on the basis of data in Table 1(*h*). The comparison of the three plants based on these partly corrected capital costs is shown in Table 1(*j*), which also shows that, based on the estimates corrected for the two errors mentioned, the cost of the two-stage plan is: (*a*) $79 000 000 more than the single-stage Conference Plan; and (*b*) almost $130 000 000 more than the New York Plan.

Third Analysis

In the second analysis, official estimates have been corrected for certain errors which seem to the writer beyond reasonable question. In this third analysis, these same errors are corrected and certain other errors are considered which the writer thinks should also be taken into account. These errors are:

(*a*) It seems certain that the extra loss in head due to the use of the water through two plants over the loss due to the use in the single plant of the single-stage plan has not been allowed for in the official estimates of the power which can be generated in the two-stage plan. A correction for this loss in the quantity of power and in the resulting net income, therefore, has been deducted in the estimates of power for the two-stage plan (see Table 1(*k*)).

(*b*) It seems equally clear that an error has been made in the estimate of the winter head in the single-stage Conference Plan. This error has been estimated by the writer at 1.5 ft (although it would probably be greater than this amount) and a correction has been made in the estimated output of power and corresponding net income for the single-stage Conference Plan (see Table 1(*k*)).

Based on the four corrections discussed, the three plans are compared in Table 1(*l*). These data show that the relative cost of the two-stage plan is: (*a*) Nearly $93 000 000 greater than the single-stage Conference Plan; and (*b*) more than $138 000 000 greater than the New York Plan.

In the three analyses previously discussed the writer has considered only the estimated cost of the water-power development, including in such cost one-half the estimated cost of works common to navigation and power. It will be noted from Table 1(*m*) that, in addition to the difference shown in the cost of power plans, there are also differences in the cost of the navigation works plus one-half the cost of works common to navigation and power, which must be added to the differences previously shown, in order to ascertain the entire difference in capitalized values.

Taking the excess cost of navigation into account (Table 1(*m*)), as well as the capitalized costs of the power plants, the difference in the complete plans for power and navigation will be as shown in Table 3. In considering the data in this table, it should be noted that in none of the reports (with the

exception of the New York Report) has interest during construction been considered; and that without definite plans of comparative construction procedure no such estimates are practicable. If interest during construction were considered, as well as depreciation and amortization of navigation works, the difference shown in Table 3 would be considerably increased.

TABLE 3.—DIFFERENCE IN VALUE OF PLANS, BASED ON CAPITALIZED NET INCOME AS PREVIOUSLY DETERMINED, CORRECTED FOR DIFFERENCE IN NAVIGATION COSTS

Item No.	Analytical comparison	DEFICIENCY IN VALUE OF TWO-STAGE PLAN COMPARED WITH:	
		Single-Stage Conference Plan (B)	Single-Stage New York Plan (C)
1	Based on official estimates (Table 1 (*g*))	$68 795 560	$124 705 920
2	(*a*) Extra cost of navigation work (Table 1 (*m*))	1 974 500	15 911 500
3	(*b*) Total difference (Items Nos. 1 and 2)	$70 770 060	$140 617 420
4	Estimates corrected for excavation and extra capacity (Table 1 (*j*))	$79 301 600	$129 686 860
5	(*a*) Extra for navigation work	1 974 500	15 911 500
6	(*b*) Total difference (Items Nos. 4 and 5)	$81 276 100	$145 598 360
7	Based on corrected estimates and heads (Table 1 (*l*))	$92 701 980	$138 357 240
8	(*a*) Extra for navigation work	1 974 500	15 911 500
9	(*b*) Total difference (Items Nos. 7 and 8)	$94 676 480	$154 268 740

It will justly be claimed that such analyses as have been made are not conclusive because there may be elements of greater importance than relative financial comparisons.

One claim of advantage for the two-stage plan is that it can be developed in two steps while the single-stage Barnhart Island development must be built in one step, and the development charge on the latter during the time necessary to develop a market may exceed the difference in first cost, $38 324 000. (See Table 1(*c*), Plan B.)

The Joint Board of Engineers may not have found it feasible to develop the single-stage Barnhart Island plan in two steps, but its members evidently did not consider the fact that the single-stage plan at Massena Point could be developed in two steps. This matter is discussed in the New York Report (pages 46-47), but is here tabulated (see Table 4), in order to show the comparison of the two two-step plans.

It is evident from Table 4 that the two-stage plan has a distinct disadvantage in comparison with the New York Plan, because Items Nos. 5, 7, 9, 13, 18, 19, 21, and 24 are uniformly to the advantage of the New York Plan as to both the first and second step.

Other reasons for favoring the two-stage plan are given in the Conference Report (see page 8) as follows:

"It will be noted, by reference to the estimates given, that the single-stage project apparently costs about $38 000 000 less than the double-stage project. The difference in cost is compensated for by the earlier date that power may be made available from the double-stage project, the extent of the economic losses to the country from the flooding of additional lands, as well as the prime considerations mentioned in paragraph 9 of the Report."

TABLE 4.—INITIAL DEVELOPMENT TWO-STAGE CRYSLER ISLAND PLAN WITH INITIAL SINGLE-STAGE NEW YORK PLAN

Item No.	Comparison	Crysler Island Plan (A)	New York Plan (C)
1	Head, in feet	24	50
2	Proposed installation, horse-power	593 000	660 000
3	Available horse-power years for sale	504 000	600 000
4	Capital cost (exclusive of navigation)	$93 423 000	$82 502 000
5	Excess in cost of Plan A over Plan C	.....	$10 921 000
6	Capital cost per horse-power year available	$185	$138
7	Capital cost advantage of Plan C over Plan A per horse-power year	.....	$47
8	Capital cost, navigation works	$46 137 000	$887 000
9	Excess cost of navigation works, Plan A over Plan C	.....	$45 250 000
	FIRST STEP (14-FOOT NAVIGATION)		
10	Power works	$93 423 000	$82 502 000
11	Navigation works	46 137 000	887 000
12	Sub-total	$139 560 000	$83 389 000
13	Difference	$56 171 000	
	SECOND STEP (27-FOOT NAVIGATION)		
14	Power works	$93 080 000	$42 796 000
15	Navigation works	42 102 000	78 040 000
16	Sub-total	$135 182 000	$120 836 000
17	Total	$274 742 000	$204, 225 000
18	Difference in total first cost	$70 517 000	
19	Capitalized difference (Table 3)	$154 268 740	
	SALEABLE HORSE-POWER YEARS		
20	First step	504 000	600 000
21	Difference	96 000	
22	Second step	1 386 100	1 392 000
23	Total	1 890 100	1 992 000
24	Difference	101 900	

(*a*) It has already been shown that while the two-stage plan will cost over $38 000 000 more than the single-stage Barnhart Island Plan and over $70 500 000 more than the single-stage Massena Point Plan, the comparative cost of the two-stage plan based on the official estimate will be about $70 770 060, and probably about $94 676 480 more than the single-stage Barnhart Island Plan, and about $140 617 420, and probably about $154 268 740, more than the Massena Point Plan. (See Table 3.)

(*b*) That the difference in cost of the two-stage plan will be in any degree compensated for by the earlier date at which power may be available is more than questionable.

The time of completion of both the first step of the two-stage plan at Crysler Island and the first step of the Massena Point Plan is estimated as five years.

The Crysler Island plans require coffer-dams aggregating about 9 000 ft in length, to be built in water up to 40 ft in depth, across the St. Lawrence River channel, and maintained during long winters with heavy ice running, and are likely to involve many contingencies and consequent delays.

The spillway and power-house at Massena Point will be founded in excavation on land "at a safe distance from the water's edge" and will be constructed "in the dry," thus avoiding the hazards of river construction. Certainly the chances in favor of early completion are much in favor of the Massena Plan.

(*c*) "The economic losses to the country caused by the flooding of additional lands" mentioned in the foregoing quotation, involves 5 760 acres extra in the United States and 9 600 acres extra in Canada, or a total of 15 360 acres, the value of which at any reasonably assumed price is entirely insignificant when the greater advantages of the single-stage development are considered.

In the Conference Report (page 4, paragraph 9), it is stated:

"If, as an alternative, the International Rapids Section be improved by a single-stage project then control of flow must be secured by operation of gates and turbines at a power plant which will develop about 2 000 000 horse power. This is not regarded favorably by the down river interests, since this single plant would serve a large and populous territory both in Canada and the United States, and operation restrictions would be difficult to impose in view of the fact that it would be a dominant interest in the territory served. Again, the additional flooding of land and the extent of the embankments, along with some adverse effects at the end of the navigation season, make the single stage project undesirable."

(*a*) It is understood that in any plan the primary control of the flow of the river will be accomplished by means of control works at Galop Island. That such control is anticipated for the two-stage plan is shown by the "Reconvened" Report (page 5, paragraph 19 (*d*)) which refers to:

"A free open channel south of Galop Island for navigation, together with a diversion channel through Galop Island capable of discharge control in the interest of both navigation and power."

Such works are also applicable to any other plan. Secondary control must take place at the power-house and spillway under any plan of development. In all cases the river must flow in its normal volume either through the turbines or sluice-gates, or both, as occasion may require. The capacity of the plant will have little bearing on the matter of control as in each case the entire river flow must be balanced between the turbines and sluice-gate discharge at all times, and such control is not influenced by the head available below the structure. This control can be accomplished by automatically controlled sluice-gates as recommended by the New York Commission.[14]

(*b*) If the proposed works are operated so as to assure normal flow of the river, "down river interests" can have no valid complaints.

It is scarcely possible that any "dominant interest" of "the populous territory" of the United States and Canada will be able to affect operating restrictions injuriously to the paramount object of navigation. If the power authorities of the State of New York and the Province of Ontario can be influenced to ignore navigation interests in favor of power users, it is safe to assume that the Federal Authorities of the United States and Canada can be relied upon to enforce all necessary restrictions.

[14] See New York Rept., p. 44.

It can scarcely be demonstrated that complicating the situation by increasing the number of power plants, which in any case must be operated in parallel, would simplify operation, management, or control.

Safety of Montreal and Communities Adjacent to Embankments

While the safety of the City of Montreal and the communities adjacent to the embankments is not specially discussed in the reports, it is incidentally referred to as a matter of importance, which it certainly is.

In all plans herein considered the normal pool level above Crysler Island will be at Elevation 242. In either of the single-stage plans, considered, the water above the proposed dam will be raised to this same elevation, which will necessarily involve longer and higher embankments.

The Barnhart Island single-stage plan will require about 20 miles of embankment below Crysler Island, as compared with about 4 miles in the two-stage plan. The depth of water against the embankment in any single-stage plan will generally be less than 20 ft, and much of it will be less than 10 ft. The maximum depth for a short distance is 40 ft. The large quantities of suitable material which must be excavated for navigation purposes, and the exceptionally large cross-section for the embankment proposed by the Joint Board gives an assurance of safety the degree of which can be increased to almost any limit desired at very reasonable cost. (The embankments shown on Plate 5 of the Conference Report have a top width of 50 ft; a water slope of 4 to 1, heavily riprapped above normal water level; a land slope of 5 to 1, with a very heavy rock-fill toe and a free-board of 20 ft.)

As to the safety of Montreal, it should be noted that if any of the dams proposed in the single-stage plan could be removed instantaneously, the water in Montreal Harbor would be raised no more than a few inches. As the level of water has frequently been raised at Montreal from 10 to 15 ft, or more, by ice jams, there can be no question of trouble of this nature.

Conclusion

In conclusion, the writer would say that if the reports and estimates considered are correct, there can be no question that the single-stage Barnhart Island Plan, is at least $70 000 000 greater in value than the two-stage Crysler Island Plan, and that the single-stage New York Massena Point Plan exceeds the two-stage plan in value by at least $140 000 000. (See Table 3, Item No. 3.)

The New York Plan, which apparently received no consideration by the Joint Board, seems to be not only much less expensive in first cost, but also much less hazardous in construction, and to be an improvement from a navigation standpoint by eliminating one lock, greatly reducing the necessary length of canal.

The writer can see no valid reasons to justify the two-stage plan which can outweigh the millions of dollars of value in favor of other plans. If the reports are in error or if the reasons for advising the two-stage plan cannot be made manifest, the writer believes that this project should be further and completely investigated before the United States and Canada are asked to make the great permanent investment involved by this project.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WATER-POWER DEVELOPMENT OF THE ST. LAWRENCE RIVER

Discussion

BY T. H. HOGG, M. AM. SOC. C. E.

T. H. HOGG,[15] M. AM. SOC. C. E. (by letter).[15a]—Before considering the paper under discussion, the writer wishes to state that in his opinion the choice of a single-stage or a double-stage plan cannot be settled satisfactorily solely on engineering or economic grounds. He would like to make his standpoint clear in regard to the entire question of the proper means of developing the International Reach of the St. Lawrence River. That part of the river under discussion forms the boundary between the United States and Canada. New York State, on the American side, and the Province of Ontario, on the Canadian side, border on the river, and by virtue of their position and ownership of the water power incidental thereto, are vitally interested in the proposed development. It is obvious, therefore, that National, State, and Provincial opinions must be given consideration. For these reasons the question of the relative suitability of a single-stage as against a two-stage plan of development is not a simple one. With these conditions in mind, a treaty, between the United States and Canada, has been negotiated, signed, and now (1933) awaits ratification.

General.—Because of the magnitude of the undertaking and its international aspects, the development of the St. Lawrence Waterway has occasioned the most widespread interest and discussion on the part of the general public and those interests particularly affected. From an engineering standpoint, it has been the subject of wide discussion, since it has occupied attention in one form or another for more than three decades. At the turn of the century, the Board of Deep Waterways issued an exhaustive report on the navigation features. Following the publication of this report, the Long Sault Development Company and the allied Canadian company—the St. Lawrence Power Company—made studies of the power possibilities

NOTE.—The paper by Daniel W. Mead, M. Am. Soc. C. E., was presented at the Joint Meeting of the Power Division of the Society with the American Institute of Electrical Engineers and the Hydraulics Division of the American Society of Mechanical Engineers, Chicago, Ill., June 29, 1933, and is published in August, 1933, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought all members for further discussion.

[15] Chf. Hydr. Engr., Hydro-Elec. Power Comm. of Ontario, Toronto, Ont., Canada.

[15a] Received by the Secretary June 29, 1933.

of the Long Sault Rapids and secured a charter to develop power at that point; but owing to the fact that its charter was found to be unconstitutional, the Company did not go on with the work.

In 1918, the Hydro-Electric Power Commission of Ontario began its studies, which have been carried on practically continuously since that date. In 1920, the International Joint Commission referred the question of improving the International Section of the St. Lawrence River to an International Engineering Board. This was followed by investigations on the part of various private interests and by New York State. It will be realized, therefore, that many plans must have been studied and discarded. This, in all likelihood, accounts for what may appear to be lack of sufficient detail in the later reports.

Naturally, when dealing with such a project, there must be differences of opinion as to the proper method of handling the problem presented, and, for these reasons, it is evident that the determination of the proper course to follow cannot be based on purely engineering grounds. Neither, when it comes to the economic side, can the intangible values (which occupy positions of widely varying importance in the view of those studying the problem), be lightly brushed aside.

In considering the entire question of the development of the St. Lawrence River, the fact that it is an international river cannot be ignored. Each country must deal with the problems that arise, in the way best suited to itself. What may appear as an obvious solution of a domestic question to one may not be at all suitable when dealt with by the other. These matters only add to the difficulty of securing a proper solution of the problem.

In his very interesting paper, the author has given a general outline of the latest studies, and has indicated quite clearly that he does not agree with the report of the Joint Board of Engineers. He also shows a marked preference for the plan of development presented by the Engineering Board of the St. Lawrence Power Development Commission for the State of New York, and uses it as a basis for comparison with other plans.

Discussion.—The project referred to as Plan A—the "Two-Stage Reconvened Plan"—is the Crysler Island Two-Stage Project, with the upper dam and power-houses at Crysler Island and the lower dam and power-houses respectively, at the upper and lower ends of Barnhart Island (see Fig. 1(*A*)). Plan B—the "Single-Stage Conference Plan" (Fig. 1(*B*))—is essentially the Barnhart Island, Single-Stage, 242 Project presented in the report of the Joint Board of Engineers. It was presented subsequently, with minor changes, in a report by the Conference of Canadian Engineers, a board made up of the Canadian members of the Joint Board and two engineers representing the Province of Ontario. Plan C—the "Single-Stage Massena Plan" (Fig. 2)—is that project which was reported on favorably by the Engineering Board of the New York State St. Lawrence Power Development Commission.

A comparison of the relative merits of a single-stage as against a two-stage project involves questions other than those of engineering import, and, since the author has indicated a certain preference, it is proposed to deal

with it in the following way: First, examine the two single-stage methods of development; and, second, discuss the reasons which led up to accepting the two-stage plan and its relative merits compared with other proposed plans.

Professor Mead has undertaken a comparison of the various plans on the basis of the estimated costs. It is of prime importance, therefore, that each estimate be on the same basis. In so far as Plans A and B are concerned, they are strictly comparable, since great care was exercised to insure that the estimates of the Conference and of the Joint Board were on the same basis. In respect to the estimates of these two projects and that of the Massena Point project (Plan C), there is good reason for taking exception to the comparison.

Capital Cost.—In Table 1(*c*), the author shows that the total capital costs of Plans B and C are $236 418 000 and $204 225 000, respectively, indicating apparently a balance in favor of the latter, of $32 193 000. An examination of the detailed estimates of the two projects discloses that the estimated cost of comparable items and unit prices are not on the same basis. It is found that $15 403 000 should be added to the estimate for Plan C for land damages, rehabilitation, etc., on the Canadian side, $1 250 000 for contingencies on "land damages", and $12 477 000 for powerhouse machinery and equipment, etc.

If the unit prices for work to be done in the same or similar locations are equalized, the estimate should be increased by $5 006 000, and a further sum of $11 340 000 should be added to cover a supply weir at the head of the Massena Canal, the excavation of an additional 5 000 sq ft in the channels between Lotus and Ogden Islands, and to provide for a double flight of locks. These additions bring the total estimated cost of Plan C to $249 701 000, as against $236 418 000 for Plan B. It is not contended that the equipment cannot be purchased or the work cannot be done for the prices given, but the foregoing adjustments are required in order that the estimates of all three plans may be on the same basis.

Head Developed.—With regard to the head that may be developed under Plan C, the official report states (page 5), that "the head will vary therefore from 84 to 88 feet, and is expected to average 85 feet". This variation in head corresponds to the fluctuation in tail-water level due to the variation in discharge, and does not take into account the fluctuation in the head-water levels due to the same cause. For a flow of 235 000 cu ft per sec, and a regulated Lake Ontario level of 246.32, the head-water level at Massena Point would be 242.15. Due to the fact that the total flow of the river will be discharged into the pool at the south end of Polly's Gut, and considering the channel improvements provided in the estimates, the water level will be from 0.8 ft to 1.5 ft higher than under natural conditions, giving a tail-water level of 156.0 and a corresponding head of 86.15 ft, instead of the 88 ft used by the author in computing the power output for open-water conditions.

The head that may be secured under winter conditions should also be corrected. The loss in head above the plant in winter, due to increased slope, will be as great as that in Plan B, and would amount to 4.3 ft. In addition. there will be a loss, due to higher tail-water levels. In an ordinary

winter, the water level below Polly's Gut averages between 10 and 15 ft above normal water level. The flow of ice will be reduced, but during the latter part of December, or early January (depending on the season), some packing will occur between the Lake St. Francis ice sheet and Polly's Gut, as this channel will always be open. Under these circumstances, a rise of from 5 to 6 ft is to be expected. The head at the Massena Point plant, corresponding to the winter flow of 210 000 cu ft per sec, would be 76.65 ft.

Power Output.—In reference to the output that may be expected from the plants, the writer is unable to agree with the figures used by Professor Mead. Primary power is continuous power and, therefore. is limited by the winter head and flow. Secondary power is that which is available in addition to primary power during the remainder of the year. Expressing the output in horse-power years, based on a winter flow of 210 000 cu ft per sec (three months), and a summer flow of 235 000 cu ft per sec (nine months), and using the corrected heads, it will be seen from Table 5(*b*) that the total horse-power years available are reduced for each of the plans, but that the greatest change is for the Massena Point project (Plan C). Instead of an output of 1 992 000 hp-yr (Table 1(*b*)), it should be 1 852 605 hp-yr, a reduction of 139 395 hp-yr (see Table 5(*b*), Item No. 9). The output for Plans A and B should be corrected by deducting 67 380 hp-yr and 68 110 hp-yr, respectively, from the figures given in Table 1 (*b*), Item No. 8.

Hazards of Construction.—The elimination of the hazards of construction in the Massena Point project, when compared with the projects proposed by the Joint Board and the Conference, has been stressed by its proponents and also by the author. An examination of the results of borings at the site proposed for the Massena Point power-houses and spillway shows that the over-burden varies in depth from 65 to 120 ft. To open such an excavation in the materials to be encountered, with adjacent water levels 65 ft above the rock-surface elevation, is certainly not less hazardous than that involved at the sites at the foot of Barnhart Island, where the over-burden is a maximum of 50 ft in depth and varies down to a few feet, and the unwatering is a very minor problem.

The main dam above Hawkin's Point, proposed in Plan C, is a rock-fill structure with a blanket of sluiced material on the up-stream side. No provision is made for diverting any of the flow while the structure is being built in the main channel for a water level above the dam lower than Elevation 200. Above this elevation the water would be diverted over the sluice-ways at Massena Point. The cribs carrying the construction trestle must be placed in water 35 to 40 ft in depth, in velocities up to 15 ft per sec, with a flow of from 215 000 to 280 000 cu ft per sec; and, since it is not proposed to clear off the rock surface, there is likelihood of underscouring. Not only is this proposal very hazardous, but there is great uncertainty as to the quantity of material required in the blanket because no one can foretell what slope the materials will take when placed under water by sluicing methods.

The location of the concrete dam proposed in Plan B is such that at least 70% of it can be built in still water, a great part of which is quite shallow, and where the rock is lying at a depth varying from 15 to 25 ft below the water surface. By building this section of the dam up to the surface of the river bed, the remainder can then be put in by diverting a large part of the flow, after which the entire dam can be raised a section at a time to its final elevation. This proposal is considered by those intimately acquainted with the river, as much less hazardous than the one suggested by the proponents of Plan C.

Summary of Comparisons of Single-Stage Plans.—To summarize, then, a comparison of the single-stage plans shows that, after the estimates are made strictly comparable, the capital cost of Plan B is $13 283 000 less than that of Plan C. Correcting for slope and loss in the tail-race, the summer heads on the two plants for a flow of 235 000 cu ft per sec would be 86.1 and 86.2 ft, respectively, and, in winter, for a flow of 215 000 cu ft per sec, the heads would be 76.4 and 76.65 ft, respectively. In respect to hazard, Plan B is more free from that objection than Plan C, and the quantities, especially in the dam, are capable of being estimated much more closely. From the Canadian standpoint it is more satisfactory because the Canadian power-house can be placed on Canadian territory.

In the report of the Board of Engineers, New York State, it is stated that "they [the Board] do not feel that so strong a position can be taken as to the superiority of the Massena Point one-stage plan over other single-stage plans." Apparently, the plan was finally presented for consideration, not because its authors were convinced of its overwhelming superiority, but as an alternative which might help in a final determination of a satisfactory scheme of development. The report states:

"It is appreciated that the choice of the exact location of the dams and power houses must be determined along with other negotiations which, of necessity, remain to be made with the Canadian interests."

Comparison of Plan A with Plans B and C.—In considering the two-stage Crysler Island plan contained in the "Reconvened Report," it is pointed out that to state "the Canadian engineers have consistently advocated a two-stage plan of developing the International Rapids Section," is erroneous. In the first report the Canadian engineers were not convinced it was desirable or feasible, on account of ice conditions, to raise the pool level at Barnhart Island to Elevation 242.0, but they favored a single-stage plan with a lower level, namely, 232.0, with a control dam at Ogden Island. In presenting the Ogden Island two-stage, as an alternative to the single-stage plan, the Canadian engineers were endeavoring to meet the wishes of the power interests, to make available for power as much of the total fall in that section of the river as possible.

Following the 1921 report, both the Hydro-Electric Power Commission of Ontario and the Department of Railways and Canals (which organization secured all the field data on the Canadian side and carried on the office investigations for the Canadian representative) made further intensive studies of the

winter conditions on the river. The viewpoint of the Canadian Government officials was that, with the open-water reaches that would exist under any single-stage plan studied to that time, serious ice jams were certain to occur, similar to those conditions which have been observed from Weever's Point up stream under natural conditions. The down-river interests looked on these possibilities with great apprehension; they considered that it would inevitably lead to disaster and insisted that in any plan of development their interests must be amply safeguarded.

Ice Conditions.—It is emphasized that the seriousness of the ice problem in relation to the development of the St. Lawrence River cannot be too greatly stressed. Under natural conditions there is a stretch of river, more than fifty miles long, in which ice in its various forms—board, frazil, anchor, and slush—is formed on a colossal scale. This accumulating under the ice sheet at the upper end of Lake St. Francis has caused shoves and packing to occur all the way from Summerstown up stream to Lock 21, and on numerous occasions it has resulted in raising the water level at the foot of Polly's Gut and the lower end of Barnhart Island from 15 to 25 ft above normal level. Jams have formed below Weever's Point, which have caused the ice to pack up stream to Morrisburg, resulting in a rise in water level of 12 to 15 ft at the latter point. Study of the river during the winter season has engendered a wholesome respect for the problem of providing safe winter operating conditions.

Reasons for Crysler Island Two-Stage Plan.—In order that the objections of the down-river interests might be met, the Canadian officials proposed a control dam at Ogden Island, so that the flow could be throttled if a jam began to form. This led to a demand on the part of the power interests of Ontario that the control dam should be designed to develop power.

To raise the pool level to Elevation 242.0, without disturbing present lake levels, requires very heavy channel improvements from the head of Galops Rapids down stream. Coupled with the studies of winter conditions, it was considered that the only safe way to deal with the situation was to enlarge the channels until an ice cover could be secured, and to make provision for the control of the flow to prevent ice gorging, since the velocities could not be reduced in the entire reach to the point where ice would form in place, as the formation of an ice cover is dependent to a considerable extent upon the packing of ice carried in from the open reaches above.

While these studies were being made, the communities on the Canadian side of the International Rapids Section of the St. Lawrence River were deeply interested and, through their parliamentary representatives, discussed the effect to them of the different methods of development proposed; so there was built up, through various organizations, a tangible and strong public opinion for the adoption of a two-stage plan. The expressed public opinion finally had to be given serious consideration. The grounds for the stand taken by these communities were as follows.

The north shore of the St. Lawrence River constitutes the oldest settled section of the Province of Ontario, and, in many cases, the land is still held and occupied by descendants of the original holders. This area was known

as "the Front," from which the back townships were settled; and the river towns and villages were, and still constitute, the marketing and trading centers for a distance half-way to the Ottawa River. The objection of these communities to disturbance is strenuous and continuous, so much so that it has come to be recognized by both the Provincial and Dominion Governments; strengthened by the objections of the down-river interests, it is a real and insurmountable obstacle to securing the single-stage plan. It may be pointed out that Plan A, the two-stage plan, has also been objected to on the score of disturbance, and while there are some grounds for these objections, still the single-stage plan would result in disturbance to, and the elimination of, the entire tier of communities from above Cornwall to the head of Galops Rapids, a distance of forty miles; in the two-stage plan, on the other hand, the disturbance would be confined to one-half that distance and would not extend any appreciable distance back from the river.

In a two-stage plan, with the upper plant at Ogden Island taking the place of a control dam at that point, an ice cover could only be secured below the plant by very heavy work in the channels as far down stream as Crysler Island for any water-surface elevation below 225.0. After numerous borings had been made to determine the elevation of the rock surface, it was decided that the proper position for the upper plant in a two-stage plan was at Crysler Island. This was the reason Plan A was finally proposed by the Canadian engineers; and the Joint Board agreed that, "all things considered, the scheme was feasible from an engineering standpoint."

The foregoing is a brief outline of the steps followed by the Canadian engineers in reaching the position as to the desirability of a two-stage plan, and indicates the engineering problems.

Table 5(*a*) gives a comparison of the three plans for the heads which would obtain under winter and summer conditions. In regard to the power produced under the three plans of development, each is expressed in horsepower years. With the corrected heads and using 85% efficiency in each case, Table 5(*b*) gives a comparison of the power output under the three methods of development.

Comparison of Plans of Development on Basis of Capital Cost.—Table 5(*c*) gives a comparison of the three plans on the basis of capital cost after they have been placed on a comparative basis by corrections to the unit prices and other items, as outlined in a preceding paragraph. The corrections to the estimates of Plan C do not entirely adjust the differences. When studying the plan of the New York State Board after the report was presented, the layout shown therein was accepted, but the quantities were taken off and estimates were prepared in the same way as for Plans A and B. It was then found that the capital cost was $273 822 000 as compared with $236 418 000 for Plan B. In view of this, it was considered that Plan C was distinctly inferior to other single-stage plans that had already been studied.

In Table 5(*d*), the costs are subdivided in order that the annual expenses may be calculated. The values for Plan C are corrected in accordance with the revised capital costs for that project. Table 5(*e*) shows the estimated annual expenses which would be incurred in the operation of the three projects.

TABLE 5.—WRITER'S COMPARATIVE DATA PERTAINING TO PLANS A, B, AND C

Item No.	Description	Plan A; Two-Stage, Crysler Island, "Reconvened" Plan		Plan B; Single-Stage, Barnhart Island, Conference Plan		Plan C; Single-Stage, Massena Point, New York Plan	
		Winter	Summer	Winter	Summer	Winter	Summer
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	(*a*) COMPARATIVE HEADS, IN FEET (SEE TABLE 1 (*a*))						
1	Project elevations (see Fig. 3)	217	217	242	242	242	242
2	Head at Crysler Island	19.6	24.2	.....	.....	.....	.....
3	Head at Barnhart Island	56.4	60.4	76.4	86.1	.....	.....
4	Head at Massena Point	.....	.....	.....	.....	76.65	86.15
5	Total head	76.0	84.6	76.4	86.1	76.65	86.15
6	Reduction in head in winter	8.6	.....	9.7	.....	9.5	.....
	(*b*) POWER OUTPUT (SEE TABLE 1 (*b*)), IN HORSE-POWER YEARS						
7	Primary power	1 539 125		1 547 225		1 552 290	
8	Secondary power	283 595		303 265		300 315	
9	Total power	1 822 720		1 850 490		1 852 605	
10	Excess power, Plans B and C, over Plan A	.....		27 770		29 885	
	(*c*) COMPARATIVE ESTIMATED COSTS (NO INTEREST DURING CONSTRUCTION; 27-FOOT NAVIGATION (SEE TABLE 1 (*c*))						
11	Works primarily for power	$132 452 000		$87 481 000		$93 627 000	
12	Works common to power and navigation	108 102 000		121 196 000		129 094 000	
13	Works solely for navigation	34 188 000		27 741 000		26 982 000	
14	Total	$274 742 000		$236 418 000		$249 703 000	
15	Excess cost of Plans B and C over Plan A	.....		$38 324 000		$25 039 000	
	(*d*) COSTS SUBDIVIDED TO COMPUTE ANNUAL EXPENSES (SEE TABLE 1 (*e*)).						
	Upper Pool; Power Works:						
16	Depreciable	$49 736 000		.....		.....	
17	Non-depreciable	5 769 000		.....		.....	
	Upper Pool; One-Half Common Works:						
18	Depreciable	6 764 000		.....		.....	
19	Non-depreciable	31 154 000		.....		.....	
20	Sub-total (Items Nos. 16 to 19)	$93 423 000		.....		.....	
	Lower Pool; Power Works:						
21	Depreciable	$64 695 000		$78 719 000		$86 216 000	
22	Non-depreciable	12 252 000		7 717 000		7 411 000	
	Lower Pool; One-Half Common Works:						
23	Depreciable	7 093 000		11 336 000		15 180 000	
24	Non-depreciable	9 040 000		49 262 000		49 367 000	
25	Sub-total (Items Nos. 21 to 24)	$ 93 080 000		$147 034 000		$158 174 000	
26	Total (Items Nos. 20 and 25)	$186 503 000		$147 034 000		$158 174 000	
	(*e*) ESTIMATED ANNUAL EXPENSE (SEE TABLE 1 (*f*) AND TABLE 5 (*d*) (ITEM NO. 26))						
27	Interest at 5%	$9 325 150		$7 351 700		$7 908 000	
28	Maintenance and operation	1 598 000		1 579 000		1 579 000	
29	Depreciation (1.25%)	1 603 600		1 138 750		1 267 450	
30	Sinking fund (1.053%)	1 963 875		1 559 270		1 665 570	
31	Total annual costs (Item Nos. 27 to 30)	$14 490 625		$11 628 720		$12 420 020	
32	Difference between cost of Plan A and Plans B and C, respectively	.....		$2 861 905		$2 070 605	
	(*f*) COMPARISON OF POWER PLANT COSTS BASED ON CORRECTED CAPITAL COST AND EQUALIZED INSTALLED CAPACITY						
33	Estimated capital cost for power	$186 503 000		$147 034 000		$156 099 000	
34	Equalized installed capacity	2 200 000		2 200 000		2 200 000	
35	Total cost of output (horse-power years)	1 822 720		1 850 490		1 852 605	
36	Excess output, Plans B and C over Plan A	.....		27 770		29 885	
37	Capital cost per horse-power year: {Item No. 33 / Item No. 35}	102.37		79.40		84.26	
38	Excess in favor of Plans B and C (cost per horse-power year)	.....		22.97		18.11	
39	Total income	25 922 800		26 241 000		26 287 500	
40	Total annual charges	14 490 600		11 628 700		12 420 000	
41	Net annual income (Item No. 39 minus Item No. 40)	11 432 200		14 612 300		13 867 500	
42	Excess annual incomes, Plans B and C over Plan A	.....		3 180 100		2 435 300	
43	Difference capitalized at 5%	.....		63 602 000		48 706 000	

These are based on the experience which the Hydro-Electric Power Commission of Ontario has had in operating its plants. The rates used are neither the highest nor the lowest, but are considered to represent, fairly, the operation costs of these plants. Item No. 28, Table 5, includes management, engineering staff, superintendence, record staff, stenographers, telephone operators, operating payroll and supplies, maintenance and repairs, liability insurance, and the operation and maintenace of the dams.

The estimated cost of maintaining and operating the plants is given in Table 6.

TABLE 6—ESTIMATED OPERATING COSTS (SEE TABLE 2)

Item No.	Description	CRYSLER ISLAND TWO-STAGE 217 PROJECT				BARNHART ISLAND SINGLE-STAGE 242 PROJECT		MASSENA POINT SINGLE-STAGE PROJECT	
		Upper Plant		Lower Plant					
		Operation	Maintenance	Operation	Maintenance	Operation	Maintenance	Operation	Maintenance
1	Operating staff (engineering)	.....	.....	$30 000	.....	$30 000	.....	$30 000	.....
2	Superintendent, assistant superintendent, record staff, stenographers, telephone operators, etc.	.....	.....	30 000	.....	30 000	.....	30 000	.....
3	Operation payroll and supplies	$52 000	.....	238 000	.....	280 500	.....	280 500	.....
4	Maintenance and repairs	.....	$102 000	.....	$272 000	.....	$374 000	.....	$374 000
5	Total	$52 000	$102 000	$298 000	$272 000	$340 500	$374 000	$340 000	$374 000
6	Total operation and maintenance	$154 000		$570 000		$714 500		$714 500	
7	Total operation and maintenance: two-stage, four plants	.....		1 448 000		.....		.....	
8	Total operation and maintenance: one-stage, two plants	.....		.....		1 429 000		1 429 000	
9	Management	.....		150 000		150 000		150 000	
10	Total	$154 000		$1 598 000		$1 579 000		$1 579 000	

A comparison of the power plant costs of the three plans, based on corrected capital costs and equalized installed capacity is given in Table 5(*f*). This is parallel to the author's comparisons presented in Table 1(*l*). If the differences between the net income from the single-stage plans (B and C) and Plan A are capitalized at 5%, the results compare with those computed by Professor Mead, as follows:

Reference	Plan B	Plan C
Table 5(*f*), Item No. 43.....	$63 602 000	$48 706 000
Table 1(*l*), Item No. 94.....	$92 701 980	$138 357 240

It is confidently asserted that in so far as Plan C is concerned, the tremendous difference in capital value presented in Table 3 would vanish entirely if, as was indicated in an earlier paragraph, the plan were placed on the same basis as the others as regards safety, freedom from

operation hazard, and navigation facilities, and if the questionable proposal of placing a rock-fill structure in a river of the nature of the St. Lawrence and at the point proposed were eliminated.

Reasons for Not Accepting Author's Set-Up.—For the reasons given in the discussion of capital costs, no further reference has been made to Professor Mead's "First Analysis" of the three plans, but what is considered to be the correct comparison is given in Tables 5 and 6. It is proposed now to give the reasons for not accepting the corrections made by the author in his second and third analyses. Before doing so, attention is called to the fact that the Canadian organizations studying the problem have been living with the river for a number of years, and that practically the same personnel has been engaged on the work throughout. While these organizations have been working independently, they have conferred and discussed the problems freely. Furthermore, the Hydro-Electric Power Commission of Ontario, has discussed these questions with organizations from the United States side of the line, and has furnished, freely, whatever information they might desire. On the United States side, while the personnel has not been so continuously on the problem, the United States Government organization (the Army Engineers, and, through them, the Lake Survey) has been engaged on the river more or less continuously for a number of years.

In contrast to these conditions, the New York State Engineering Board reported in a little more than three months after its organization and, as a Board, never had the opportunity to study winter conditions, first hand, which is probably the greatest problem of this development. Surely then, its plan should scarcely be used as a measuring stick to judge the other methods of development from the point of view of cost, etc. The author has acknowledged that, in making the analysis, he has been handicapped by his lack of knowledge concerning local conditions, and his difficulties are fully appreciated. The writer wishes to emphasize his belief that the plan recommended, and on which the treaty between the United States and Canada has been negotiated, has been arrived at after careful and long study, and, all things considered, represents the best solution of the problem.

Necessity for Increased Cross-Section in the Single-Stage Plan.—In his capital cost comparisons (Table 1(*c*)), Professor Mead has added the sum of $4 149 000 to Plan A to provide an additional 5 000 ft to the cross-sectional area in the reach between Lotus and Ogden Islands, similar to that in Plan B. This addition is not considered necessary. In explanation, the additional cross-sectional area was considered necessary in the single-stage plan to reduce the danger from ice gorging. This danger is greater under the single-stage plan for the following reasons: The control dam at the head of the Galops Rapids is situated in an artificial channel and provides only partial control. Due to changes in atmospheric pressure, surges are set up on Lake Ontario which will cause a rise in gauge height of 2 to 3 ft in a few hours at Prescott, or at the head of Galops Rapids, resulting in a relatively material increase in slope in the pool above the power plant, with a consequent increase in velocity and flow.

Occurring at a time when the ice on the pool is forming, as these surges frequently do, could result in the formation of an ice gorge, with serious results to the power output during the remainder of the winter season. These surges can be anticipated, and in Plan A there is time, by closing the sluice-gates in the upper dam, to raise the pool level and counteract the increased slope; and, at the same time, by transferring part of the load to the lower plant, it is possible to reduce or eliminate any reduction in power output during the period that the flow is being manipulated. In the single-stage plan, with its much larger pool, it would be impossible, in most cases, to reduce the slope in time to meet the emergency. It was considered that the extra cross-sectional area would provide the additional safeguard to reduce this hazard.

Heads at Crysler and Barnhart Islands.—It is not considered that the head on the Crysler Island plant should be reduced by from 1.3 to 1.5 ft (Table 1(*k*)), any more than the heads on the plants at Barnhart Island or Massena Point. The loss through the plant is taken care of in the efficiency used in computing the power output. The necessity of providing an extra 5 000 sq ft in the channels below Lotus Island has been explained. The apparent excess loss in head of 1.5 ft on the Barnhart Island plant in winter (Table 1(*k*), is due to the extra accumulation of ice in the tail-race in any single-stage plan.

Power Costs, Barnhart Island.—If the installed capacity of the plants at Barnhart Island and Massena Point is equalized, the cost of the Barnhart .Island development should only be reduced by the cost of the electrical equipment, since there would be no reduction in the spacing of the units, or in the size of the turbines, to take care of the 87 700 hp, considering the number and size of the units. This reduction in cost would not amount to more than $1 045 000.

Navigation Costs.—With reference to the navigation costs, the estimates in the Massena Point plan provide for a single flight of locks on Barnhart Island; the fact that the tonnage capacity was not the same in each case, was not considered. In both the other plans, the capacity of the locks is 16 000 000 tons, while that of the single-flight locks in Plan C is only 12 000 000 tons. In order that the capacity may be increased, the estimates were increased to provide for double-flight locks.

Comparison of Initial Developments.—The author has compared the upper development of the two-stage plan with the first-stage of the Massena Point project (Table 4), and, while of interest, it is not expected now that the need will arise for such a program of construction. In considering the relative output of the Crysler Island plant and the Massena Point initial development, it is contended that all the 600 000 hp available from the latter plant will be prime power; in other words, the head of 50 ft will be available throughout the year. For an elevation of 210 at Massena Point, the upper end of the pool would be about at Morrisburg. With the natural slope in the river above that point, there would be twenty miles of open water from which ice would be carried down into the pool during the

entire winter. This would certainly result in an increased slope and reduced head-water level. The tail-water level would be increased by the accumulation of ice in the channel below Polly's Gut.

It is estimated that during the latter part of December, and during January, February, and the early part of March, the head will be reduced by 20 to 25 ft in an ordinary winter, so that the net head on the plant would be between 25 and 30 ft, instead of the 50 ft assumed in Table 4. The net head on the Crysler Island initial development—owing to the fact that natural levels would obtain below the plant—would be from 28 to 30 ft. The power output of each plant is also incorrect. The available continuous power from the Crysler Island plant would be 568 000 hp, and from the Massena Point plant, 506 000 hp, or 62 000 hp less than from the two-stage Crysler Island plant.

The capital cost of the initial development of the Crysler Island plant is given as $93 423 000, or $185 per available hp, in Table 4. This capital cost includes one-half the cost of works common to power and navigation, amounting to $37 918 000. A large part of this sum is chargeable to channel improvements, which are as valuable to the Barnhart Island plant as to the Crysler Island plant. It has been considered as fair that only one-third the cost of channel improvements above the plant should be charged against the upper plant, and the remaining two-thirds against the lower plant. This division is on the basis of installed capacity, approximately. The capital cost, therefore, of the Crysler Island plant, as given in Table 4, should be reduced by $14 360 000.

The total capital cost of providing 25-ft navigation in the upper pool is charged against the initial development of the two-stage plan, although Table 4 includes the sub-title, "First Step (14-Foot Navigation)", while, in the Massena Point plan, only the cost of providing a lock for the continuance of the present 14-ft navigation is included. The cost of providing similar navigation facilities in Plan A is included in the estimates of the cost of works common to navigation and power for the upper pool, and amounts to $1 205 000. The total shown in Table 4 to cover the cost of navigation, therefore, should be omitted. With these corrections, the capital cost of power for the Crysler Island initial development becomes $79 063 000, and that of the initial Massena Point development becomes $89 284 000, and the corresponding cost per available continuous horse-power would be $139.25 and $176.50, respectively.

In Table 3, Professor Mead summarizes his comparison of the three plans, and the conclusion he reaches is that Plan C is from $140 600 000 to $154 268 000 better than Plan A. It has been shown that these comparisons do not give the proper picture because compared with Plan B, when the difference in income is capitalized, the balance against the two-stage plan is $63 600 000 (see Table 5(*f*), a fact that has always been known and taken into consideration. In so far as Plan C is concerned, instead of being superior to Plan B by $60 000 000 (Table 3), on the basis of capitalized earnings, it is inferior by about $14 900 000, leaving

out of consideration the fact that from an engineering standpoint its proponents were not at all certain that it was superior to Plan B, and that there is good reason to doubt that it is as free from construction hazard.

The author has made reference to the probable effect on the water level in Montreal Harbor if, by some mischance, the dikes were breached. His statement that, in such an event, the level would only be raised by a few inches is undoubtedly correct; but he has lost sight of the fact that in Quebec, above Montreal, there are industrial centers and areas that are not in the fortunate position of Montreal. In the event of such a disaster, they would suffer great damage, and there is the possibility that a new channel might be created for the St. Lawrence south of the new Beauharnois Power Canal. These communities cannot help but be apprehensive, and their apprehension takes the form of serious objection through their political representatives.

Conclusion.—In conclusion, it is pointed out that the comparison drawn, using Plan C as a basis, is incorrect, because the plans as presented in the official reports are not comparable.

Revising the estimates of the official report by using the same unit prices and adding certain items that are omitted, but not altering the quantities, would bring them more nearly in line with those of Plans A and B. The capital cost of Plan C would then be (Table 5, Item No. 14) \$249 703 000, instead of \$204 225 000 used by the author in Table 1(*c*). If the estimates of quantities and costs are prepared in the same way as those of Plans A and B, the capital cost would be \$273 822 000.

As between Plans C and B, it has been shown that when placed on the same basis, Plan B is distinctly superior as regards total capital cost, cost of power, revenue, annual charges, freedom from construction and operation hazard, and capacity and superiority of navigation works. If objection is taken to placing the American power-house on Barnhart Island, it is quite possible, with little increase in cost, to place a concrete dam below the head of Barnhart Island, alter the navigation route by leaving the river above Robinson's Bay, build twin-flight locks at the mouth of Grass River, and place the American power-house on the main shore below Hawkin's Point, where the rock is much higher than at the proposed Massena Point site.

Considering hazards of construction, a careful analysis of the proposed methods of construction and a knowledge of the physical conditions, the hazards in the two-stage plan are not as great as those to be expected in the Massena Point plan. The capital cost is practically the same when the estimates are prepared on the same basis, and, in addition, there are those advantages in its favor which are associated with reduced disturbance and satisfied communities below the developments.

A comparison of Plan A and Plan B indicates that, both from the standpoint of capital cost and the annual costs which would be chargeable against power, the latter is preferable, although both plans are sound from the standpoint of the engineering problems involved and from the broad economic viewpoint. After years of study of all aspects of the development

of the International Rapids Section of the St. Lawrence River—during the course of which it has been abundantly evident that there are a great many factors of other than an engineering, or a purely economic, character which have had a profound influence upon the problem, and to which as a consequence it was necessary to give due weight—Canadian engineers have reached the conclusion that the two-stage Crysler Island plan is the most suitable method of meeting, satisfactorily, all the varying conditions involved.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

THE MARTINEZ-BENICIA BRIDGE

Discussion

BY W. H. KIRKBRIDE, M. AM. SOC. C. E.

W. H. KIRKBRIDE,[9] M. AM. SOC. C. E. (by letter)[9a]—The writer is indebted to those who have discussed his paper, particularly for the additional information presented.

Mr. Helmers calls attention to the use of triangulation in measurements and layout. The site of the bridge is such that direct measurements were difficult, because of the depth of water, the strong currents, and large volume of water traffic. He also mentions some of the difficulties encountered in sinking the cylinders to retain the sand of the sand islands. It must not be assumed that sinking caissons by this method is free from difficulties, but the ingenuity shown in straightening the cylinders and caissons after they had settled out of level, and in maintaining the caissons in a vertical position, resulted in landing them quite close to their true position and well within the tolerances allowed.

Mr. Clements gives additional information and points out the advantages of the use of the "sand island" method, in which the writer fully concurs. He directs attention to the fact that while the use of artificial islands is not new, the use of the rigid steel shell to enclose sand to form the artificial islands, is new. The writer recognizes this fact in his Conclusion (2), in which he referred to "artificial islands of sand within steel shells." Credit for the use of this method is due to Mr. Clements, who, as Consulting Engineer for the Contractor, was responsible for the suggestion that the method be used, and for working out the details of construction.

Mr. Schneider calls attention to the "proof-testing" of all the heat-treated eye-bars used in the bridge. When the specifications for the superstructure were under discussion, heat treatment of eye-bars was rather new and to be certain that all bars were of equal quality and conformed to the requirements, arrangements were made to "proof-test" each bar. The eye-bars before proof-

NOTE.—The paper by W. H. Kirkbride, M. Am. Soc. C. E., was presented at the meeting of the Structural Division, Sacramento, Calif., April 24, 1930, and published in September, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1932, by Messrs. N. F. Helmers, E. J. Schneider, and M. F. Clements; and February, 1933, by Jacob Feld, Assoc. M. Am. Soc. C. E.

[9] Chf. Engr., S. P. Co., Pacific Lines and Subsidiary Lines, San Francisco, Calif.

[9a] Received by the Secretary May 31, 1933.

testing had their pin-holes bored to a diameter sufficiently smaller than their final diameter so that after testing they could be re-bored to exact length and pin-hole diameter, thus removing any permanent set. The proof-testing consisted of applying to the bars a load of 36 000 lb per in. of nominal cross-section.

The re-boring of the pin-holes was supposed to take care of only the slight distortion that might occur. Bars with any permanent set in the body of the bar were to be rejected. It is gratifying to know that all the bars proof-tested were accepted; they were all tested to destruction and complied fully with the specifications.

Mr. Feld gives valuable information in listing examples of deep caisson construction, and calling attention to the progressive development during the past ten years.

Mention was made in the paper of earthquake faults near the bridge and of efforts to design the superstructure and foundation to resist earthquake vibration. Since the completion of the bridge, the United States Coast and Geodetic Survey has placed a strong-motion seismograph on top of one of the piers with the expectation that the record obtained may be of assistance to engineers in developing more accurate principles as to the design of bridge piers to resist earthquakes. Similar machines have been installed in various places in California. Some difficulty has been experienced in designing a starting device that will be sensitive to slight earthquake shocks, and at the same time, not respond to other vibrations, such as may be produced by the starting and stopping of trains on the bridge, but it is believed that this trouble will soon be eliminated.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

Discussion

BY ROBERT FLETCHER AND J. P. SNOW, MEMBERS, AM. SOC. C. E.

ROBERT FLETCHER[97] AND J. P. SNOW,[98] MEMBERS, AM. SOC. C. E. (by letter).[98a]—A gratifying interest is revealed by the number of discussions submitted in connection with this paper. These discussions have brought to light historical matter of great value, and have increased the knowledge of timber bridges still doing good service. They have widened the view of the ancient history and of the later European period. That timber bridges have yet a "reason to be" is amply proved; and their use for some decades seems to be possible. These fruitful results of a suggestion made by other members of the Society constitute an important chapter of the history of the profession, which might have been lost beyond recovery.

Concerning Cæsar's military bridge—to meet the objection that the notched blocks, as the only connectors at the ends of the bents, might fail by crushing at the sharp edges, or splitting between the notches—a model of one span on two bents was made on a scale of one-twelfth that of the prototype. An estimated load on the span to include the weight of the large, rough-hewn green timber and a column of troops, four abreast, carrying arms and accoutrements, was computed to exceed 14 000 lb. By scale reduction the equivalent load on one bent of the model should be in excess of 100 lb (1 to 144). The model sustained two heavy men aggregating nearly 400 lb in weight, on two bents; that is, nearly 200 lb on one bent. After separation, the parts gave no indication of over-strain.

NOTE.—The paper by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E., was published in November, 1932 *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: January, 1933, by Henry B. Seaman, M. Am. Soc. C. E.; February, 1933, by Messrs. Jasper O. Draffin, E. K. Morse, and John W. Storrs; March, 1933, by Messrs. C. J. Hogue, William A. Oliver, Philip G. Laurson, Richard S. Kirby, Wells N. Thompson, Benjamin Wilder Guppy, Ivan E. Houk, and William H. Baker; April, 1933, by Messrs. Oren Reed, R. P. Davis and L. L. Jemison, Peter T. Landsem, J. K. Finch, A. A. Eremin, Aksel Anderson, and William G. Atwood; and May, 1933, by Messrs. H. Fugl Meyer, W. E. Howland, Harry K. Ellis, and Nelson J. Bell.

[97] Director Emeritus, Thayer School of Civ. Eng., Dartmouth Coll., Hanover, N. H.

[98] Cons. Engr., Boston, Mass.

[98a] Received by the Secretary June 7, 1933.

Figs. 46 and 47 are two notable views of Timothy Palmer's Bridge at Easton, Pa. (see Fig. 11), taken after the bridge had been in continuous service for 89 years. These photographs were taken in 1894 by Eugene Mowlds, M. Am. Soc. C. E., just one year before the removal of the bridge to make way for a structure capable of sustaining traffic by trolley cars. These views of the exterior are valuable as showing the care which the master builder exercised to produce a distinctive architectural effect, in contradistinction to the barren, cheap-as-possible housing since adopted in covering wooden bridges. Fig. 46 shows the appropriate covering with pleasing lines, harmonizing with the surroundings and giving a sturdy aspect to the structure as a whole. Fig. 47, a view of the Easton end, shows enough adornment to relieve homeliness; and, above the portal, is the name and date to serve as the signature attesting the authorship.

Under the heading "The Howe Truss," the writers mention as unusual types, two Howe truss bridges with an arch on both sides of each truss, and state that they had no knowledge of any other Howe truss bridges so constructed. In correspondence with the writers, F. J. Nevins, Valuation Engineer, of the Chicago, Rock Island, and Pacific Railroad Company, and Frank F. Fowle, of Frank F. Fowle and Company, Consulting Engineers, of Chicago, Ill., have stated that the first bridge over the Mississippi River was built for the Chicago, Rock Island and Pacific Railroad Company, between Rock Island, Ill., and Davenport, Iowa, during the years 1853 to 1856. It consisted of five fixed spans of Howe trusses, each about 253 ft, with arches on both sides of each truss, and a draw-span. They also state that a second bridge was built a few years later on the same masonry, of Howe trusses having the unusual feature of curved upper chords, and no auxiliary arches. These are historic bridges because of the strenuous and prolonged opposition of the steamboat owners up and down the river to their construction, and because the unbroken railroad connection thus made across the river gave a great impetus to the settlement of the States beyond.

Questions listed in a letter by J. R. Worcester, M. Am. Soc. C. E., with appropriate answers by the writers, are as follows:

Question 1.—Have not arches been found beneficial to abutments, as well as to trusses, in furnishing lateral pressure where most needed?

Answer.—An arch, of course, tends to assist an abutment in resisting the pressure from the filling at its back, but it should not be depended upon to save a weak abutment. An instance is recorded in which a bulging abutment led to collapse of the arch, breaking it at the crown, and buckling the truss. On the other hand, a case is cited of a T-abutment with narrow stem, in which buttress masonry behind the heads was needed when arches were added to the bridge.

Question 2.—What is the reason for the different thicknesses of planks shown in Fig. 17?

Answer.—It is wholly to facilitate construction. The customary way to start an arch is to set up light supports on the falsework to define the curve

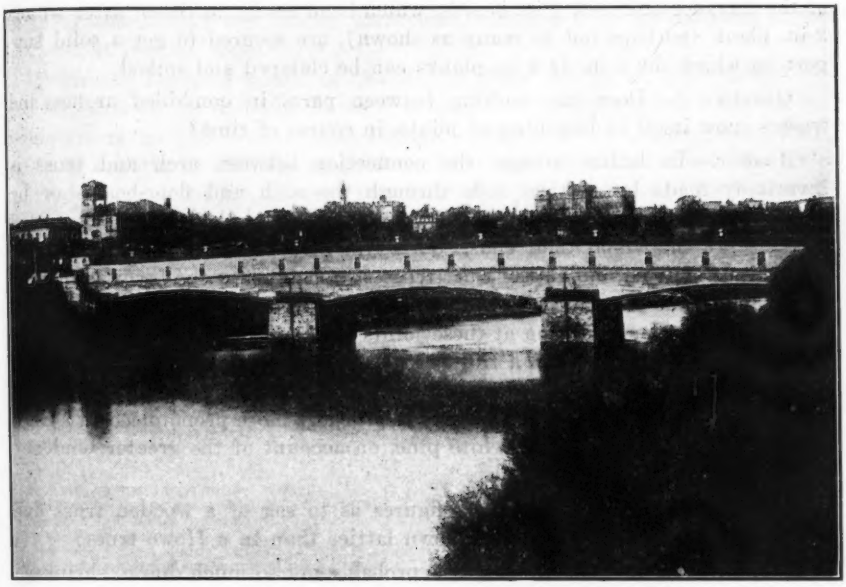

FIG. 46.—BRIDGE OVER DELAWARE RIVER, AT EASTON, PA.

FIG. 47.—ENTRANCE TO BRIDGE OVER THE DELAWARE RIVER, AT EASTON, PA.

of the intrados and tack ⅞-in. boards, which bend easily, to them; after which, 2-in. plank (perhaps not so many as shown), are secured to get a solid support on which the 3-in. or 4-in. planks can be clamped and spiked.

Question 3.—Does the working between parts in combined arches and trusses show itself in loosening of joints, in course of time?

Answer.—In lattice bridges the connection between arch and truss is invariably made by vertical rods through the arch and floor-beams or by needle-beams below the lower chords. Any slackness in these rods is corrected by the screw-threads at their ends. This type of connection is advisable with other kinds of truss, but in the Burr trusses of early days the favorite method was to frame the posts to make seats on the arch. The flexibility of timber is such that no serious working at these joints has been observed by the writers. The creaking observed when a train crosses one of these bridges is normal to wooden bridges. It may be caused by warped sticks coming to a full bearing when a load passes over the bridge. It is probably more pronounced in spruce bridges than in those built of white pine, on account of the greater tendency of spruce to warp.

Question 4.—Can you give any figures as to sag of a wooden truss due to shrinkage, and is this less in a Town lattice than in a Howe truss?

Answer.—Sag in wooden bridges is probably not so much due to shrinkage as to compression of surfaces in contact at joints. Lattice trusses without arches naturally sag more than Howe trusses on account of the great number of joints composed of wooden pins in bored holes. This involves pressure on the rough side-grain of pins as well as a tendency to deflection of the pins. To counteract this tendency it is good practice to build 50% more camber into lattice trusses than into those of the Howe type. Howe trusses, if sagged, can be adjusted by setting up their vertical rods. In the lattice type the sag is cumulative. The unique bridge at North Blenheim, N. Y., shown in Fig. 20, has held its camber from 1855 to the present, although it has no vertical truss rods. This is due, without doubt, to the care used in its framing and the seasoning of the timber before framing as noted by Mr. Morse in commenting on the Ninth Street Bridge, in Pittsburgh, Pa.

Question 5.—What units have you found safe for stresses in bolts and tree-nails in shear? For direct tension and horizontal shear in spruce and yellow pine? Also, in fiber stress in stringers?

Answer.—As is shown in Mr. Guppy's discussion the size of members depends in some cases on connections rather than on longitudinal stress. Unit stresses that have been found, by experience, to result in good practice are listed in Table 1.

Bridges built to these units have carried loads, the computed stresses from which are twice these units, without showing any ill effects.

Mr. Seaman shows a 2-track jack-knife draw (Fig. 30). The birth and culmination of this type of draw have been shown[99] in *Engineering News-Record,* by a single-track patent drawing and a photograph of the 8-track

[99] *Engineering News-Record,* Vol. 108 (1932), p. 93.

draw of the Boston and Maine Railroad Company, over the Charles River channel, at Boston, Mass.

Professor Draffin's notice of the Enoch Hale Bridge is appreciated. It was the first in the United States, that could be called a framed bridge, its predecessors having been of the stringer type, none of greater span than could be negotiated with the magnificent pine spars then available. Mr. Hale secured money from Mr. Geyer, a merchant of Boston, mentioned by Professor

TABLE 1.—ALLOWABLE STRESSES IN TIMBER

Description	Yellow pine	Spruce	Oak pins	Iron
Tension	1 000	800	.....	.....
Compression	800	650	.....	.....
Bending modified by factor containing depth ÷ span	1 200	1 200	.....	.....
Bearing end-grain	1 000	800	.....	.....
Bearing side-grain, whole width of timber	350	300	.....	.....
Bearing side-grain, small area like washers	400	350	.....	.....
Shearing, longitudinal	80	80	.....	.....
Bearing	.....	.....	400	.....
Shearing	.....	.....	600	.....
Bending	.....	.....	1 800	.....
Floor-beam hangers; tension, net section	.....	.....	.....	7 500
Lower chord coupling rods; tension, net section	.....	.....	.....	10 000
Lower chord gib-plates; bending	.....	.....	.....	12 500
Arch suspender rods; tension	.....	.....	.....	10 000
Lateral rods, loaded chords; tension	.....	.....	.....	15 000
Lateral rods, unloaded chords; tension	.....	.....	.....	20 000

Draffin, by a mortgage on his toll bridge. It was a well-paying piece of property which Mr. Geyer had been trying to buy for several years. The mortgage was on short time, and, when its expiration approached, Mr. Hale succeeded in borrowing from his friends to liquidate it. He sent the money to Boston by a messenger who dallied a day at Lowell, and when he tendered the funds to Mr. Geyer, was told that he was a day too late, that the mortgage had expired, that Mr. Geyer had already assumed the title, and that the bridge was his. Such was the law of foreclosure in those days, derived from the Cavalier laws of England, the result being that Enoch Hale lost his noteworthy bridge and became indigent in his old age.

Professor Draffin states in his first paragraph that the present concrete successor of the Hale Bridge was built in 1930. The previous lattice bridge was removed in 1930 on account of failure due to a decayed wall block, but the handsome concrete structure was completed and dedicated with fine ceremony in 1932.

The writers concur in Professor Draffin's conclusion that repairs rather than rebuilding were done by Mr. Geyer when the Enoch Hale Bridge was eighteen years old; and that it was done as window dressing in connection with obtaining a mortgage instead of for strengthening the structure. In fact, there is a book extant containing a picture of the village and landscape made from a point at a considerable distance from the bridge, showing its braced stringer outline with the spandrils filled with lattice trim. Probably this trimming with new fences and floor were the extent of Mr. Geyer's repairs. The statement made by President Dwight about the bridge being an obtuse arch has misled several writers. A braced stringer bridge of that magnitude

is comparable to a segmental arch of three voussoirs which make obtuse angles with each other. Fig. 8 is an excellent line drawing of the oil painting made soon after the bridge was erected, but the braces were much more sharply inclined than shown, and the right-hand end of the floor should be raised to continue the grade of the other end to meet the side of the gorge higher than indicated. The highway along the mountain side was much higher than the bridge, so that a steep descent was required.

In speaking of the Tucker Bridge (Fig. 18), Professor Draffin calls it of the lattice timber type. This is technically wrong. It was a single-plank lattice bridge of 3 by 10-in. plank, while Fig. 19 is a timber lattice of 6 by 8-in. timbers. Much more of historical interest could be written of this bridge and its contemporary events than is pertinent.

It is gratifying to read in the discussion by Mr. Morse, his appreciation of the splendid workmanship in the bridges built by Theodore Burr. The writers refer to the same characteristic in those built by Timothy Palmer; and indeed it was commented[20] on by the late Theodore Cooper, M. Am. Soc. C. E.

The writers join Mr. Storrs in honoring the early builders who had little to guide them in planning or building their bridges, except their experience and strong common sense. These men and the early builders named in the paper were real engineers. The historians of America have generally ignored them, perhaps because they were regarded as mere carpenters. They were among the "empire builders." They are more deserving of large mention in history and inscriptions on memorial tablets, than many who have achieved less for their country. They are comparable to the "Brothers of the Bridge" who built and maintained the great stone bridges of Europe during the Middle Ages.

The writers can not entirely subscribe to the enthusiasm expressed by Mr. Hogue in the second paragraph of his discussion, as to a recrudescence of timber construction in the country at large; but in regions of primitive growth, as claimed in the paper, they believe a considerable field is open for wooden bridges. The examples of longevity cited for treated lumber (40 years), seems absurd when applied to truss bridges. If properly covered with roof and side boarding 100 years is frequently attained. An example is cited later in this discussion of one, 120 years old, which is still "going strong."

The ring dowels for timber connections mentioned by Mr. Hogue are certainly worthy of consideration. Astonishing examples of their use are illustrated in recent publications of the National Committee of Wood Utilization, U. S. Department of Commerce, Washington, D. C.

The paragraphs of the main paper describing Fig. 16 answer Mr. Oliver's uncertainty in regard to the writers' conclusions as to the adaptability and economy of wooden bridges. Mr. Oliver does not mean that the Selby Bridge, built in 1790, is still in use, as to its original timbers. Stringers bridges last

[20] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 1.

from 5 to 20 years, depending on latitude, character of lumber, and exposure to the weather, but their members are readily renewed, and hence the bridge, as an adjunct of transportation, lasts perpetually. Preservative treatment will double the durability of their members.

Regarding Pons Sublicius, mentioned by Mr. Oliver, the writers do not think any published picture of its construction is authentic.

The writers are greatly indebted to Professor Laurson for the items relating to early European bridges, which he cites. It is interesting to learn that lattice bridges were built on various railroads in England and Ireland previous to 1846. Ithiel Town's patent for this type of bridge was granted in 1820, and it is on record that he visited England about 1840. No doubt he was successful in selling rights to build by his patent. Regarding "assembling" bridges on land before erection in place: If Professor Laurson means assembling them erect, it is seldom, if ever, done. The only practical way to build lattice bridges and others, such as the Burr and Long types, in which the web members pass through the chord, is to frame the trusses lying flat on skids on shore, or on falsework, if wide enough, and traffic is suspended. Holes for pins and bolts are bored in this position and, when complete, each stick is carried to its place on the falsework and assembled erect. The chords of Howe trusses are framed on the falsework and the trusses erected complete after the lower chords are in place. The practice has been to fabricate and erect these bridges as one job; whereas in metal bridges the two phases of the work are widely separated. This distinction may disappear in the Machine Age, supplemented with new methods of making timber connections; but it is incredible that the building of wooden bridges should become common in regions where organized fabrication shops are within economic reach of the locus of the bridge.

A wrong impression is conveyed by Professor Kirby's reference to the span of the Schaffhausen Bridge (Fig. 7). It consisted of two spans of 171 and 193 ft. The writers have no data showing the span of the McCalls Ferry Bridge, but mention several of much greater span than the Schaffhausen, including some built previous to 1814. (See Wernwag's "Colossus" of 340-ft span, built in 1812 (Fig. 12), and one at Wittingen, Germany, 390-ft span, built during the third quarter of the Eighteenth Century.)[17] It is an interesting fact that a complete model of the Schaffhausen Bridge is still treasured in the town 134 years after the destruction of the bridge by a French army. This is without the covering and exhibits all the details of construction.[100]

Fig. 32, presented with the discussion by Mr. Thompson, is very interesting. The weather protection mentioned by him can be seen to consist of a kind of flexible shingle applied to each stick of the corbels and cross-ties. The presence of this bridge in far Japan, coupled with the numerous examples of the same type now existing in Northern India, and the Savoy ruin, mentioned in the paper, under "The Ancient Background," seem to prove that

[17] From Gauthey's Atlas accompanying his 3-vol. treatise on "Constructions des Pont."

[100] An excellent view of the model appears in Mohringer's "Bridges of the Rhine", pub. in 1931.

the corbelled bridge was known and used across the great Eurasian Continent in the early ages of human development.

Mr. Guppy calls attention to an interesting and true phase of the difference in designing wooden and metal bridges. The comparison might be extended to a somewhat similar difference in fabricating and erecting the two classes. Fig. 23, or its modification, might well be added to Figs. 16 and 28 as examples of the use of metal splices in timber tension members.

The simple method of anchoring a bridge to its abutments described by Mr. Houk, is well worth using on all wooden bridges over streams subject to high water or severe winds. Hundreds of wooden bridges have been floated —pushed by drift or blown by wind—from their bearings, nearly all of which would have escaped serious injury had they been anchored. It seems scarcely necessary to carry the rods to the upper chord. A short rod each side of the lower chord, passing through a yoke block, and each one connecting with two wire lines below, would seem simpler to install than the arrangement described by Mr. Houk.

The combination wood and steel bridges described by Mr. Baker are interesting examples of history repeating itself under new conditions. Mr. Baker's type of construction suggests great possibilities for highway bridges of spans up to 80 ft. Lower chords of channels in one length up to two cars long, with perhaps a plate reinforcement at the center, with angle-blocks and tie-plates riveted on at the shop, or laminated chords of treated plank securely united as one timber with ring dowel, or other connectors, would naturally follow as a type of construction suggested by Mr. Hogue. Preservative treatment is not needed if the ends are protected from dirt accumulations, and if treated wall-blocking is used. Rod verticals and timber upper chords and web diagonals may be used. All spans greater than 45 ft should be housed with roof and side covering, using a roof with a good overhang and a pitch of about 4 in. per ft. Inexpensive trim on facade and sides should be used, and the entire structure painted appropriately to present a bungalow appearance, far surpassing in picturesqueness those of the New England covered type. Shorter spans with lower trusses should have each truss housed, roof and sides. A concrete floor-slab on steel cross-floor-beams would prevent the decay complained of, and housing will save the end posts and upper chords. Window openings (see Fig. 13), admit light and increase safety of traffic inside, better the appearance outside, and assist ventilation. The flooring, of whatever character, furnishes adequate lower lateral bracing. The upper chords should be adequately braced in connection with the roof framing.

The writers disagree with Mr. Baker in his statement that wooden bridges are very costly to maintain. It is inferred that the tendency to decay is the cause of his complaint. Piles in sea water, infested with marine insects, open stringers, and plank floors, and, in fact, all timber exposed to the weather, or in contact with masonry or earth, are short-lived in warm climates unless thoroughly treated with a preservative. Truss bridges on the contrary are very durable if protected by housing. (Note the many instances cited in the paper and discussions of bridges a century and more old.) An experience of more than forty years has taught the writers that current maintenance of

wooden truss spans is not so costly as maintenance of steel spans. The worst enemy of wooden bridges is fire, due at present largely to cigar and cigarette stubs. Obsolescence and increase of loads, especially in case of railroads, are the usual cause of renewals, rather than physical defects.

Mr. Reed and Messrs. Davis and Jemison have furnished valuable records of early truss bridges west of the Allegheny barrier. These trusses as a whole are based on the Burr design, a worthy example to follow.

The Ramp Creek Bridge built about 1837, with its arches, upper chords, and roof framing of yellow poplar (white wood), and its floor-beams, stringers, and under floor of walnut, cause surprise to an Eastener; but it really shows excellent engineering judgment on the part of its builders and was not less remarkable than the use of the magnificent white pine for bridge work in the East, which timber was once reserved, by law, for masts of the Royal English Navy. The fact that after about 95 years of service the bridge was deemed worthy of re-erection in a location where it would be subject to no heavier traffic than that for which it was built, is certainly worthy of attention.

Failure of floor-beams by longitudinal shear is common if they are overloaded and mortised near their ends for lateral braces, as described for the bridges in Figs. 35 and 37. A well-laid floor is very ample for lateral bracing of a highway bridge. The "bowing" out of line mentioned in the case of the former bridge is quite likely due to flatness of arch and lack of rigidity of the shallow truss with no counters.

The durability of the hand-forged bolts mentioned in the description of this bridge is worthy of note, although not strictly pertinent to a paper on wooden bridges. The more nearly pure an iron is, the better it resists corrosion. Old-time Swedish or Russian iron, or present-day ingot iron, is practically rust proof; while structural steel is not so durable as wood in certain locations.

The Kennedy family, J. J. Daniels, and Lemuel Chenowith, should be added to the honor roll of names of noted bridge builders.

Another variation of the Burr type of truss was seen in a bridge built by George W. Thayer, of Massachusetts, over the Delaware River, at Narrowsburg, N. Y., in 1859, which had the longest framed trusses (clear span of 262 ft), except Wernwag's "Colossus" (see Fig. 12), of which the writers have knowledge.[101] Each truss had two arches on each side, one above the other, separated by 12 in., or more. A peculiarity in the trusses was horizontal struts ("straining pieces") between the posts, fixed just below the upper, and above the lower, chords where the diagonal braces were framed into the posts. The latter feature was specified in a patent granted to Mr. Thayer in 1845.[102] This and other patents noted in the paper are illustrated and described in a series of articles,[103] by F. B. Brock, Patent Attorney, Washington, D. C., on expired patents for wooden, iron, and combination bridges. The Narrowsburg Bridge was removed when it was about forty years old, while still capable of giving good service (probably for more than twice that period).

[101] *Engineering News*, Vol. 42, p. 122.
[102] *Loc. cit.*, Vol. 9, p. 418.
[103] *Loc. cit.*, Vols. 9 and 10.

The "modern" timber connectors shown by Mr. Landsem are interesting. The multitude and variety of these patented connectors reminds one of the flood of United States patents issued prior to 1880 for bridge trusses of wood, iron, and combinations of the two, each patentee claiming that his arrangement of parts was safer and cheaper than any previously proposed system. The upper chord joint (Fig. 41), seems like a substitute for the American pin-joint. By hooking the four members to a ring in this joint there is no moment induced on the center bolt. Its workmanship must be quite expensive. On this account it is not probable that this joint will be generally used in this country. There is a wide field for their use as smaller connectors in high towers of timber trestles, towers for transmission lines and radio aerials, and especially for portable structures, such as oil-well drilling derricks, observation stands, etc. The old-time rectangular key, with a length of about ten times the depth of dap cut in the timber for it, and two bolts to resist the overturning tendency, is ideal in efficiency, but is perhaps not so economical in these days of machine employment as the circular dap and connector. A simple disk with a central bolt is probably the best solution of the problem.

Mr. Landsem has contributed some valuable facts as to European bridge building, ancient and modern. His statement about the flexibility of laminated arches should not deter builders from their use. If properly secured to stiffening trusses they hold their shape perfectly. Mr. Landsem mentions the materials as being "boards." Arches are built up of 2 to 4-in. "boards" very readily. Radial bolts should be inserted at intervals of not more than 8 ft, and the load applied at points should not be more than 12 ft apart. If greater intervals are allowed buckling may occur.

Professor Finch contributes a fine historic summary—a splendid frame in which wooden bridges occupy but part of the picture. There is little in it for the practical bridge builder to discuss, but the writers feel disposed to comment upon the feature of statical indetermination and the men named as bringing American practice to rational analysis.

It is a fact that the two all-wooden types best adapted for bridges are inherently statically indeterminate; namely, the Burr (Figs. 14 and 15), and the lattice with supplementary arch (Fig. 17). The elasticity of the material, the tolerance of timber joints to slight distortion, and the natural law of gravitation will bring the stress to the most rigid members. Timothy Palmer's Easton bridge (Fig. 11), was a rather indifferent truss yoked to a flat arch for a lower chord. Theodore Burr improved on this, introducing more indeterminacy but also a large amount of practicality, resulting in wide adoption and approval, as shown throughout this paper and its discussion. The lattice with arch is superior to the Burr type in that it requires much simpler forms of timber, much less skilled labor, and its truss is much better adapted to resist reversals of strain. Still the rank indeterminateness is there, and no one but the schoolmen know or care anything about it. If there is strength enough in both members of the combination acting together, the elasticity and safe "give" at joints will distribute the load exactly according to the static rigidity of each.

Regarding the men named by Professor Finch: Colonel Long was a bridge builder of courage and imagination, although his designs were not so extensively copied as were those of Burr, Town, and Howe. Neither Ithiel Town nor William Howe were builders of bridges. Town was a business man who patented the lattice type which was designed and built in Vermont years before the patent was issued. Sanford Granger, mentioned in the paper, was probably connected with Mr. Town, in securing the patent. Mr. Granger, however, did not invent the type. A lattice bridge was built in 1813 (when Granger was only 17 years old) over Otter Creek, in Pittsford, Vt., many miles from his home. The bridge is still in service after 120 years. It was thought worthy of being floated up river to its place in 1927 when the high water of that year carried it a mile or more from its abutments. Town also patented the timber lattice in 1839. This is also of Vermont birth. William Howe was an architect. In 1839, in connection with Amasa Stone, then 22 years old, he built a bridge of 75-ft span for the City of Springfield, Mass., probably on the Howe principle, because in 1840 he secured a patent, and they built the great bridge over the Connecticut River (Fig. 24), as stated in the paper. The next year Stone bought out Howe's rights in the patent and went into bridge building as a business. Later, he formed a company under the name of Stone and Boomer which engaged in railroad and bridge construction on a large scale.

Concerning the reference to Trajan's Bridge over the Danube River by Mr. Eremin, the figures quoted are those given by Nicholas Bergier[16] and Dion Cassius.[104] The latter was the Roman historian whose notable history of the Roman Empire in eighty books was written between 217 and 229 A. D.—the century following Trajan's death in 117 A. D.[105] The historian's account of this prodigious and unprecedented work states that it was built to facilitate Trajan's second campaign against a revolted enemy, who killed himself after complete defeat in 106 A. D. The same authority puts the building of the bridge about 104 A. D. The brief existence of the bridge is also noted, for the narrator adds that Trajan's successor, the Emperor Hadrian, caused the arches to be torn down because he was extremely jealous of the fame which this 'wonder of the world"—as the narrator styles it—had brought to his predecessor.[106]

The information on foreign bridges contributed by Messrs. Eremin and Andersen is highly appreciated. The double-web systems of the Gulfoss and other bridges of the Howe type are interesting. They admit the use of smaller diagonals and rods than single systems but, for spans such as those mentioned, it is doubtful if the added work of framing and assembling did not more than balance the saving of material. The omission of side boarding seems wasteful. The lower chord needs protection from the elements more than the upper one. The type of trestle shown was formerly used in some

[16] "Histoire des Grandes Routes de l'Empire Romain", pub. in Brussels, Belgium, in 1728.

[104] Appleton's Cyclopedia, Vol. VI, Article on Dion Cassius, p. 122.

[105] *Loc. cit.*, Vol. XV, Article on Trajan, p. 838.

[106] Bergier's "Histoire", as before, Tome 2, livre 4, p. 750

sections in the United States and has the advantage, if properly designed, of allowing the removal of any piece of timber for replacement without interrupting traffic.

Northern Russia and Scandinavia with their wealth of coniferous timber growth are the natural homes of the lattice type of bridge. Baltic fir is well adapted for that type if proper units of stress are used. One-inch round iron or steel drift-bolts, instead of the oak pins shown on Fig. 16, would be advisable for connectors.

The suggestion of Mr. Atwood that early obsolescence may overtake a structure built for all time is of much greater moment than many realize. Most engineers have dug out so much that their predecessors built under the slogan, "build for the future"—good, durable, and expensive work no doubt, but ill adapted to present-day needs—that they should feel that their work will probably go the same way, and that they should build simply what is needed now, and pay for it. The other alternative is to do as has been done many times, to wit, build for the future, paying for it with bonds for coming generations to liquidate long after the constructions have been abandoned or dismantled.

Mr. Meyer presents another specimen of the interesting type of corbelled bridges which have appeared at several points in this paper and its discussions. The great heart of the Eastern Continent lying east of the Ural Mountains and including Western China, in which the Kingdom of Muli is situated, has much to reveal of the early history of Aryan peoples. This type of bridge, called "cantilever" by Mr. Meyer, must be accepted as the prototype of constructed bridges. It is found from the Pacific to the Atlantic Oceans, coming to Europe, apparently, with the Alpine stock of people in very ancient times. Its class had better be called "corbelled," leaving "cantilever" to the distinctive type with the back lever doing duty by spanning an opening.

Professor Howland submits some appropriate information about the engineering literature available in the Eighteenth and early Nineteenth Centuries. It recalls research made for the paper, the results of which were precluded because of limitations of space. The lack of suitable timber in England, and the resort to cast iron by the English engineers was noted in the paper under "Later European Timber Bridges." For example, this appears by consulting the monumental folio atlas accompanying Rickman's "Life of Telford." That portrays, by fine engravings, 22 stone bridges, including masterpieces such as the famous London bridges; 35 road bridges in the Highlands of Scotland—from arched culverts to those of two or more spans—all of stone; 8 bridges built with cast iron, made in sections and bolted together—two over the Thames and others less known; 2 famous suspension bridges; and only 2 timber bridges of minor importance.

Turning to the European Continent a different story comes to light. The treatises on "The Arts of Construction and Carpentry" by French engineers—Gauthey, Bélidor, Bruyére, Cordier, and especially E'my—are prolific in fine drawings of actual or suggested timber bridges in a great variety of arched and braced contrivances. Perronét, who made his great stone bridges patterns (as it were) for English engineers and the world to follow, developed the

designing—as Professor Howland has described—of the heavy timber centers which he required. It is in order to add that the splendid atlas of plates portraying his works has only one drawing of a timber bridge, and that was a "project" for a bridge over the Seine, in Paris, with seven arches of 60-ft span and 10-ft rise, on piers of timber piles. Evidently, he preferred to build for posterity.

Mr. Ellis corroborates others in showing the general prevalence of the Burr type of bridges in certain sections of the country. In some others, particularly in Northwestern New England, the Town lattice prevailed. Burr's designs were not conspicuous for scientific proportioning of parts. His experience and natural flair for heavy construction were probably his only guides. The later changes in character of highways and of loads carried, would find weak spots in many of his bridges, especially those built by others who had not his flair and who built at his type rather than to it. The combination truss and arch, which is the real essence of the Burr patent as finally adjusted, is the absolute final acme of design in wooden bridges of considerable span.

Mr. Ellis adds cumulative evidence to the value of effective covering which is stressed strongly in the main paper.

Mr. Bell's discussion opens an interesting line of conjecture. He states that the bridge at Camp Nelson, near Lexington, Ky., was built by Lewis Wernwag in 1838, and he feels certain that the one at Cheat River (Fig. 35), was constructed by the same builder. In 1855, Nicholas Powers built the North Blenheim Bridge (Fig. 20), using the same principle of construction—an arch of three rings on the center line of the truss—but he carried his middle truss to the ridge pole of the pitched roof, which brilliant idea, was missed by Wernwag, both at Cheat River and at Camp Nelson. Powers adopted the later type of Long's patent truss which is better adapted for stiffening the arch than the Burr type. The arches of these three bridges follow those of Wernwag's famous "Colossus" in design, but in these later designs he adopted the parallel chord idea of Burr for the stiffening truss instead of framing it on top of the arch (see Fig. 12), thereby giving better support for floor and roof than the original example.

Without doubt the first example of an arch on the center line of a truss was at the Cheat River Bridge, and its author was Lewis Wernwag, who died at Harpers Ferry, Va., in 1843. The two examples show that he adopted, as his later practice, the arch as the main carrier of loads, placed on the center line of the truss designed to hold it from distortion under concentrated loads, even to the extent of the twin arches embedded in a single truss, as at Camp Nelson. His arch segments were hewn to the curve of the arch. This involves the weak factor of timber (longitudinal shear) to a slight extent; and laminated arches laid up with planks 3 to 4 in. thick are now generally used. It remains to examine the truss factor to determine the best type to use for the heavy concentrated loads of the present time both for highway and railroad service. Manifestly, a truss, all the members, chords, and web of which will act in tension or compression, is ideal for the purpose. The

Town lattice fills these requirements perfectly, and hence must be given preference. It is gratifying that the idea can be traced back to the illustrious Wernwag.

The sentiment expressed so strongly by Mr. Bell that wooden bridges should not be scrapped because of obsolescence and possibly weak floors, is heartily seconded and upheld by the writers.

Considerable has been written in these discussions about preservative treatment of timber for various purposes connected with bridges. In other publications, treated timber for heavy truss bridges of long span, has been advocated so that roofs and side covering would not be needed. The advisability of the use of preservative to this extent has not been proved either by the discussions or by the experience of the writers. For trestles, small deck spans, and short spans unsuited for covering, treated timber is advisable; but for trusses of spans requiring joints in lower chords, covering is demanded. Esthetic reasons, as well as economy, require covering of trussed spans. What looks worse in a landscape than an uncovered wooden through truss bridge especially if it is black with preservative? The bridge shown in Fig. 29, for example, was rightly left uncovered for temporary use only; but its unfinished aspect shows how those designed to be permanent would look.

The general trend of these discussions amply proves the longevity of wooden bridges and justifies their use in localities where they are substantially cheaper than bridges of metal or masonry.

A chronology of the development of the types described in this history and its discussion is given herewith; the first twenty-three types, in this list, built during a period of nearly fifty years, were framed by use of tree-nails and such small hardware as could be forged by blacksmiths:

Pile-trestle and stringer bridges, crib and stringer bridges after European antecedents	From 1620 forward
Enoch Hale's braced-stringer bridge, at Bellows Falls, Vt.	1785
Timothy Palmer's Essex Merrimac Bridge	1792
Timothy Palmer's Piscataqua and Haverhill Bridges	1794
Timothy Palmer's Georgetown, D. C., Bridge	1796
Timothy Palmer's "Permanent" Bridge, at Philadelphia, Pa	1804–06
Timothy Palmer's Easton, Pa., Bridge	1805–06
Grave's (second) Connecticut River Bridge, at Hanover, N. H.	1796
Windsor, Vt., Bridge, contemporary with the Grave's Bridge	1796
Theodore Burr's Waterford, N. Y., Bridge	1804
Theodore Burr's Trenton, N. J., Bridge	1806
Theodore Burr's Mohawk River Bridge	1808
Theodore Burr's Harrisburg, Pa., Bridge	1816
Lewis Wernwag's "Colossus" at Philadelphia, Pa.	1812
Lewis Wernwag's New Hope, Pa., Bridge	1813–14

Lewis Wernwag's "Economy" Bridge	1810
Earliest lattice-truss bridge of which there is record	1813
Ithiel Town's plank lattice truss, patented	1820
Truss of Stephen H. Long, patented	1830
Ithiel Town's timber lattice truss, patented	1839
Wernwag's Cheat River Bridge, West Virginia	1834
Wernwag's Camp Nelson Bridge, near Lexington, Ky. (Standing in 1933 after 95 years. In both these bridges the arch is on the center line of the truss.)	1838
The Ramp Creek Bridge, Indiana, Burr trusses, built in (Renovated and in service, 1933, after 96 years.)	1837
The Raccoon Creek Bridge, Indiana, Burr trusses. (Still in use, 1933, after 95 years.)	1838
Brunel's experiments in England with preservatives, kyanizing, etc.	1835
Wooden lattice bridges on British railways after Ithiel Town's visit about 1840, before	1846
William Howe's patent for the Howe truss	1840
William Howe's Connecticut River Bridge, at Springfield, Mass.	1840
The Tucker Bridge, at Bellows Falls, Vt., plank lattice	1840
The trusses of Thomas W. and Caleb Pratt, patented	1844
Typical Burr truss railroad bridge (framed with white pine) at White River Junction, Vt.	1848
Howe truss bridge with double arches at Bellows Falls, Vt.	1850
The unclassified truss of Nicholas Powers, North Blenheim, N. Y.	1855
First bridge across the Mississippi River, five spans, Howe trusses with double arches, at Rock Island, Ill.	1853–56
Second historic bridge at Rock Island, Ill., Howe trusses with curved upper chords and no arches, some time before	1868
The Ledyard Bridge, at Hanover, N. H., timber lattice	1859
Howe truss bridge with double arches (12 spans) at Havre de Grace, Md.	1862–66
The gallows-frame drawbridge	1660
The A-frame drawbridge	1835
The jack-knife drawbridge	1845

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DISCUSSIONS

A PROBLEM OF SOIL IN TRANSPORTATION IN THE COLORADO RIVER

Discussion

BY MESSRS. THADDEUS MERRIMAN, AND J. C. ALLISON

THADDEUS MERRIMAN,[48] M. AM. SOC. C. E. (by letter)[48a].—The subject of this paper is of great importance, and the author has forcefully presented the situation with respect to the "bed load." In 1930, the writer suggested[49] the view referred to by the author that the suspended load is only a part of the total and that the bed load carried by a flood is proportional to the bed volume scoured out.

Further reflection during the succeeding three years and a careful reading of the author's paper, including the discussions thereon, confirms the previous conclusion of the writer that,

> The bottom load carried per unit of time is equal to the volume of material scoured out per unit of length.

This theorem is a statement of the conditions that must obtain if the river channel is to refill itself while the flood is subsiding. Otherwise, the regimen of the river would be subject to constant and progressive change in proportion to the amount by which the actual conditions differed from those of the equilibrium which the theorem defines. The stability of the regimen between Black Canyon and Yuma is, therefore, evidence of the correctness of the proposition which, in connection with soundings and under the limitations stated[49] by the writer, furnishes a means of approximating the "bed load."

The completion of the Boulder Canyon Reservoir will greatly upset the equation of equilibrium in that no new material will be available to pass on down the river. Under the new conditions of reduced flood stages and uniformity of flow the regimen of the Colorado will at once begin to show marked changes which are not readily predictable, but which will be of the nature indicated by the author.

NOTE.—The paper by S. L. Rothery, M. Am. Soc. C. E., was published in December, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1933, by Messrs. C. E. Grunsky, Ivan E. Houk, and H. M. Rouse; and May, 1933, by Messrs. William T. Collings, Jr., R. F. Walter, Harry F. Blaney, and Calvin V. Davis and Dirk A. Dedel.

[48] Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

[48a] Received by the Secretary June 8, 1933.

[49] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1142.

J. C. ALLISON,[50] M. AM. SOC. C. E. (by letter).[50a]—This paper is extremely interesting. Many articles and studies have been produced dealing with this subject. The excessive heights to which the great canals of the Imperial Valley are now elevated above the surface of the farming land, due to the deposit of silt, bear witness to the extreme importance of the problem. The financial statements of the Irrigation Districts of the Valley reveal the millions of dollars in operating cost traceable in greater part to the silt-removal items, and these tabulated millions are perhaps only a fraction of the grand total of costs to the farming community as a whole, chargeable to this one item. The writer has witnessed the progressive building up of the spoil banks along the canals with a continued sense of bewilderment as to what might be the final solution.

Exhaustive studies by the United States Bureau of Reclamation will undoubtedly reveal the best practical solution of the problem. Until these studies are completed it is difficult for an engineer to determine a plan that will come nearest to solving all the vexing questions that the "New Deal" on the Colorado River, with the Boulder Dam in partial control, has brought to the front. Already Mr. Rothery's paper has brought out what are perhaps the major questions to be solved. The writer's experience with this particular matter, prompts him to state the three major questions to be answered, all with the hope that the suggestion may be helpful in guiding the Bureau of Reclamation and the profession to the best practical plan.

I.—The greatest problem of all is the disposal of the sludge returned to the river from the partly desilted water diverted to the new canal at the proposed new Imperial Dam. In the writer's opinion, the volume of soil yet to be moved by the Colorado River below Boulder Dam can be measured only by the aproximate area of river soil deposited in the valleys of the Colorado between the mesa walls. This is limited, of course, to the uncontrolled sections of the stream bed. As the partly regulated stream seeks to establish its ultimate gradient and final position under the new operating conditions artifically set up in the creation and operation of the dam and its power plants, an unknown volume of suspended silt and bed load will continue to move down stream.

If one can picture a gigantic task of finally disposing of all this deposit through the several hundred miles of unregulated channel between Boulder Dam and Imperial Dam in generations to come, the uppermost thought in one's mind is what shall be the final resting ground for this uncalculable yardage? The answer is that there are only three ultimate places of rest large enough to accommodate the ultimate load: The Gulf of California, the Laguna Salada Basin, at the extreme lower end of the river, or the Salton Sea, in Imperial County, California. To move the sludge to any of these basins entails an expenditure of great volumes of transporting water. If the ultimate irrigation demand does not preclude this great expenditure of sluicing water, then Mr. Rothery's plan of selective separation of the lower

[50] Cons. Engr., San Diego and Los Angeles, Calif.

[50a] Received by the Secretary July 5, 1933.

third of the waters transporting the bed load, becomes one worth while studying and perfecting if possible.

On the other hand, if an economy in water is necessary in providing a sluicing head, then such a plan as is proposed by Mr. Rothery is barred from consideration. As a matter of fact, the entire plan of silt removal at the head of the canal, or at any point along the canal where the sludge is to be returned to the river bed, must be barred from consideration unless sufficient water can be supplied over and above the irrigation demand, to assure a volume and velocity sufficiently great to carry the bed load on through to one of the basins of rest mentioned.

There is apparently lurking in the author's mind the possibility that through some plan or other, these sluicing waters can be picked up and used farther down the river for irrigation below the Imperial Dam. The Mexican lands of the Lower Colorado Delta present the only area upon which this water could be used. Such a water supply, loaded to its maximum carrying capacity with bed load sand and silt, would not only be an unacceptable gift to Mexico but, if interrupted in its flow by diversion works in Mexico, would defeat the very purpose of its release at Imperial Dam.

The real answer to this particular problem lies in the determination, by the Bureau of Reclamation, of the quantity of water required annually to clear the river channel of its accumulation between Imperial Dam and the basin of final rest for the material. The surplus of water available during the early years of operation, before the period of full irrigation demand, cannot be considered and counted on too seriously in the design of the desilting works. The permanent volume of water available for sluicing purposes over and above the ultimate irrigation demands must be the quantity counted on to rid the river channel of its accumulated volume of sludge, because the bed load of sand picked up below the Boulder Dam and contributed to the channel by the drainage areas of Arizona and California tributaries to the river, will supply a load to be transported to the canal works of the Imperial Dam long after the peak irrigation dam is reached.

The plans for Boulder Dam provide a continued firm horse-power output of about 663 000 which will produce the 4 330 000 000 kw-hr of prime energy contracted by the Government for sale. Based on 83% efficiency and 10% maximum shortage, as estimated by the Bureau of Reclamation, this output will require an average volume ranging between 14 000 cu ft per sec and 17 500 cu ft per sec diverted continuously through the Boulder Dam power plant. According to the writer's calculations, the peak irrigation demand for the 2 100 000 acres finally to be reclaimed below the dam, will utilize this entire output from the power plants in the summer. In the winter, irrigation of the desert valleys of the Colorado River below the Boulder Dam will only use one-third of this power-development output of water; yet, unless and until other great dams in the canyons of the Colorado River are constructed, the irrigation demand cannot be equated with the power demand. Firm power in the aforementioned amounts as contracted for must be continuously developed in order to pay the costs of the dam within the 50-year

period provided by law. Therefore, two-thirds of the winter output through the power plants must be continously wasted. According to the writer's calculations, this seasonal wastage will amount to 3 544 000 acre-ft annually at Imperial Dam. The investigation being undertaken by the Bureau of Reclamation should determine whether or not this seasonal wastage is sufficient to clear the annual deposit of sludge from the river channel below Imperial Dam permanently. If it will not, the plan of utilizing the river bed as a depository for the bed-load soils taken from the All-American Canal, should be abandoned, except in the event of a re-arrangement of irrigation water delivery to Imperial Valley, as described under Heading III herein.

II.—Except for the "impatience and sentiment from the large agricultural communities of Imperial Valley," as pointed out by Mr. Rothery, it is doubtful whether the removal of all the silt in suspension will be found by the Bureau of Reclamation to be entirely justifiable, even if a plan can be devised to do so economically. When applied to the porous lands of the eastern mesa the clear water will not be as practicable, in canal operation or irrigation, as the silted water; when delivered to the lateral canals on the steep contours of the eastern and northern sections of the Imperial Irrigation System, the clear water will bring about compensative adjustments of canal grades, requiring many additional structures or concrete lining of the canals; also, more adequate drainage systems than are provided for in the existing financial plans. In the central and west central section of the Imperial Valley, where the canal grades are generally already too flat, a clear-water delivery would be extremely justifiable and beneficial if accompanied by adequate drainage provisions. A delivery of uniformly clear water for all sections of the irrigated lands to replace the silt-bearing waters abruptly, is not practical. It remains again for the Bureau of Reclamation to determine through its intensive study the extent to which the suspended soils can be removed justifiably. It appears to the writer that a less ambitious policy of suspended silt removal should be undertaken at the beginning, with provisions for later enlargement and perfection in various sections of the canal system, after a "cut and try" experience for a few years.

III.—The writer has in mind the great basin known as the Laguna Salada Basin in Mexico in the lower end of the Colorado River, as a final resting ground for the silts and bed sands of the river, rather than either the Gulf of California or the Salton Sea Basin. This selection is made because here all the sluicing waters used in clearing the channels above, can be recaptured and used for irrigation. As a matter of fact, with the establishment of a terminal reservoir at the Laguna Salada, no part of the river water will find its way to the ocean, because all of it can be recaptured through every month of the year except at times when more than 75 000 cu ft per sec are released at Boulder Dam.

This discussion is not the proper place to review the technical problems involved in establishing this reservoir, any more than to describe its characteristics briefly. At the extreme lower end of the river, just before the channel enlarges to form the upper area of the Gulf of California, the high

waters of the river sometimes overflow and back up to the northward into the Laguna Salada, an extensive basin separated from the main delta of the Colorado by the Cucopah Mountains. The southern entrance to this huge basin is narrowed down to 5 or 6 miles. The basin broadens to the north between the Cucopah Mountains on the east and the Juarez or Peninsula Range on the west, finally terminating against a high mesa formation 8 or 10 miles south of the International Boundary Line. The floor slopes from 7 ft above sea level at the south to 17 ft below sea level at the north. This basin has been studied thoroughly, and it has been found that by blocking the southerly opening with a dike averaging 25 ft high, and by tunneling through 4 miles of the mesa formation at the north, a huge reservoir can be created, with drainage wasteways both at the south into the Gulf of California and at the north into New River and the Salton Sea. When the floor of the reservoir is desalted and the area reduced and deepened to save unnecessary loss of water by evaporation, the reservoir will still have a holding capacity of from 4 000 000 to 5 000 000 acre-ft of water.

A flood channel connecting the river through its delta with this reservoir entirely by gravity can be built by the two Governments under their agreed plan of international flood control on the lower river. This can be done with far less expenditure of money than such a channel can be built to the Gulf of California either along its present broken course, or along its abandoned channel to the east. The effect of this work would be to capture and store in Laguna Salada Reservoir all the water that will otherwise waste into the ocean. After the Boulder Dam is completed, this supply of water that cannot be regulated and saved by other means will amount to about 5 000 000 acre-ft per year. This wastage must continue until other dams the size of Boulder Dam are built in the upper canyons of the Colorado, in order to equate the power demand with the irrigation demand. Should all the water be saved by the simpler process of allowing it to flow by gravity into the Laguna Salada Reservoir, many millions of dollars will be saved, as other more expensive dams above will then not be necessary to save the winter discharge through the power plants for summer irrigation needs.

By this same process all international complications can be eliminated. The reclaimed waters can be delivered into the All-American Canal for all lands elevated in Imperial Valley below the sea-level contour. In a minor degree they can be used on Mexican lands along the border by gravity, where the elevations are not higher than 15 ft above sea level. The establishment of the reservoir would probably be the controlling factor in settling the question of the quantity of Colorado River water that Mexico is to have. As a matter of fact the recapture of waters, which otherwise would run perpetually to the ocean, will yield a supply more than sufficient to fill Mexico's demand if an exchange can be made of these waters for supplies delivered to Mexican lands at some point on the border higher than the 15-ft contour.

The cost of creating the reservoir with the delivery channel into it, the waste channels at both ends, and its delivery conduit northward, including the cost of removing the salt from the floor of the basin, could be met through a charge of 25 cents per acre-ft for the water delivered from this

storage through a period of 50 years. With its creation, no economy in the use of channel-sluicing waters at any time of the year would be necessary at Imperial Dam. As the floor elevations are raised through the deposition of silt, in the reservoir, the water level would be raised correspondingly. Due to the excess grades in the river channel constructed to the reservoir, this raising could go on continuously by raising the channel banks and the supporting ground along the entrance channel with the silt dredgings taken from the channel. The very size of the reservoir itself, almost completely surrounded by mountains and high mesa plateaus, affords a suitable resting ground for Colorado silt, in all the years to come.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MODEL LAW FOR MOTION OF SALT WATER THROUGH FRESH

Discussion

BY G. H. KEULEGAN, ESQ.

G. H. KEULEGAN,[12] ESQ. (by letter)[12a].—The derivation of model laws in certain cases of fluid motion is effected readily by means of the energy equation. The motion of salt water through fresh presents a clear example. The method, in this particular case, has the advantage of directness and affords a more intimate understanding as to the restrictions to be imposed on the phenomena for a workable model relation than the method of dimensional analysis, for example. This understanding no doubt can also be derived by the method of differential equations; but if that method is used, the rigorous computations are somewhat involved, since it is then necessary to use separately the Eulerian and Lagrangian equations expressed differently for the two liquids, with the additional boundary conditions to be satisfied at the points where these liquids are in contact. The authors use the Eulerian equation in the first stages of the motion (Equations (1) and (9), inclusive). They then revert to the Lagrangian equation, applying it only to the salt water, whereas for a rigid solution of the problem, they should use also a similar equation for the fresh water, together with the boundary conditions existing at the surface of contact. The derivation here presented corrects this defect in the derivation of the authors. It differs from that of the paper only in the definitions of the effective kinematic viscosity and the effective salinity occurring in the expressions of the model law.

Certain assumptions are necessary in the present derivation to obtain the law practically in the same form as that given by the authors. It will be assumed first that the motion is essentially a two-dimensional one. In other words, the width of the channel is of the same order as its depth, and the major part of the dissipation of energy arises from the sharp velocity gradient

NOTE.—This paper by Morrough P. O'Brien and John Cherno, Assoc. Members, Am. Soc. C. E., was published in December, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1933, by W. E. Howland, Assoc. M. Am. Soc. C. E.; April, 1933, by Messrs. Herbert D. Vogel, and C. E. Grunsky; and May, 1933, by John B. Drisko, Jun. Am. Soc. C. E.

[12] Associate Physicist, Bureau of Standards, U. S. Dept. of Commerce, Washington, D. C.

[12a] Received by the Secretary March 25, 1933.

at the boundary separating the two liquids. In this connection it is scarcely necessary to state that the question of surface tension does not enter, since the two liquids mix. On the other hand, the mixing at the surface of separation due to diffusion takes place at a much slower rate than the velocities involved in the flow of the water. Since the motion is assumed to be two-dimensional, unit width of the channel will be assumed in making the energy computations. Other assumptions will be stated as occasion arises.

In addition to the notation of the paper, ρ_1 and ρ_2 denote, respectively, the densities of the salt water and of the fresh water. Suppose that at the time, $t = 0$, the lock holding the salt water is opened and the salt water (shown

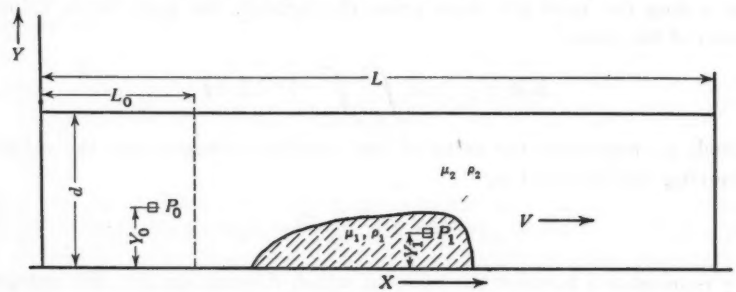

FIG. 17

as the cross-hatched area) has advanced at the time, T, to the position shown in Fig. 17. At the point, P_1, an element of salt water has the mass, $\rho_1\, dX\, dY$, and the elevation, Y_1. Initially, this element occupied the position, P_0, in the lock at the time, $t = 0$, and had the elevation, Y_0. Accordingly, the system suffers a decrease of potential energy,

$$dW = g\,(Y_0 - Y_1)\,(\rho_1 - \rho_2)\,dX\,dY \qquad (20)$$

arising from this transfer. Thus, at the instant, T, the entire system has suffered a loss of potential energy of amount,

$$W = g\,(\rho_1 - \rho_2) \int_{\text{vol.}} (Y_0 - Y_1)\,dX\,dY \qquad (21)$$

Here the integration is carried out for the region occupied by the salt water. Using the theorem of the mean, Equation (21) becomes,

$$W = g\,(\rho_1 - \rho_2)\,\alpha\, d \int_{\text{vol.}} dX\,dY$$

in which $\alpha\, d$ represents the space average of the drop, averaged over the initial salt-water volume. It follows that,

$$W = \alpha\, g\,(\rho_1 - \rho_2)\, d^2\, L_0 \qquad (22)$$

since the volume integral is equal to the volume of the lock. It is to be observed that α as derived herein is a pure number having a value which depends on T. Practically, however, α changes very slowly with time. The authors have shown that initially it has the value, 0.25.

This decrease in the potential energy of the system is compensated in part by the increase in the kinetic energy of the system. The remaining part represents energy that is dissipated during the time interval, T. In the computation of the kinetic energy and of the dissipation as well, the effect of the vertical component of the velocity will be assumed to be negligible. With this assumption in mind, the kinetic energy of the system at the time, T, is,

$$KE = \tfrac{1}{2} \int_0^d \int_0^{L_1} \rho V^2 \, dX \, dY$$

in which, V is the velocity at any point along the channel at the time, T. Since ρ does not have the same value throughout, one may write, using the theorem of the mean:

$$KE = \tfrac{1}{2} \gamma \rho_m \int_0^d \int_0^{L_1} V^2 \, dX \, dY \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which, ρ_m represents the value of the densities averaged over the volume of the moving liquids; that is,

$$\rho_m = \frac{\rho_1 + \rho_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

and γ represents a number the value of which depends on T. The integrated dissipation function, on the other hand, is,

$$2F = \int_0^T \int_0^d \int_0^{L_1} \mu \left(\frac{dV}{dY}\right)^2 dX \, dY \, dt$$

in which, $2F$ represents the energy of dissipation.[13] Since the viscosity does not have the same value throughout, one may write, using again the theorem of the mean:

$$2F = \beta \mu_m \int_0^T \int_0^d \int_0^{L_1} \left(\frac{dV}{dY}\right) dX \, dY \, dt \quad \ldots\ldots\ldots\ldots (25)$$

in which, μ_m represents the value of the viscosities, averaged over the volume of the moving liquids; that is,

$$\mu_m = \frac{\mu_1 + \mu_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

μ_1 and μ_2 being the viscosities of the salt water and of the fresh water, respectively, and β representing a number the value of which depends on T. Although rigorously β and γ are dependent on time, their variation with time will be small when the differences, $\mu_1 - \mu_2$ and $\rho_1 - \rho_2$, are small.

The principle of the conservation of energy gives,

$$\alpha g (\rho_1 - \rho_2) \, d^2 L_0 - \beta \mu_m \int_0^T \int_0^d \int_0^{L_1} \left(\frac{dV}{dY}\right)^2 dX \, dY \, dt$$
$$- \tfrac{1}{2} \gamma \rho_m \int_0^d \int_0^{L_1} V^2 \, dX \, dY = 0 \quad \ldots\ldots\ldots\ldots\ldots (27)$$

[13] Lamb's "Hydrodynamics," Fourth Edition, p. 549.

Introducing the initial velocity, V_o, of the front of the salt-water wave and writing:

$$x = \frac{X}{L_o}\,;\ y = \frac{Y}{d}\,;\ \tau = \frac{t}{T};\ L_1 = m L_o;\ v = \frac{V}{V_o}\,;\ \nu = \frac{\mu_1 + \mu_2}{\rho_1 + \rho_2}\,;\ \text{and } s = \frac{\rho_1 - \rho_2}{\rho_1 + \rho_2},$$

the energy equation (Equation (27)) becomes,

$$2\,\alpha\,g\,s\,d - \beta\,\frac{V_o^2\,T\,\nu}{d^2}\int_o^1\int_o^1\int_o^m\left(\frac{dv}{dy}\right)^2 dx\,dy\,d\tau - \gamma\,\frac{V_o^2}{2}\int_o^1\int_o^m v^2\,dx\,dy = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The corresponding equation for the model is,

$$2\,\alpha'\,g'\,s'\,d' - \beta'\,\frac{V'^2_o\,T'\,\nu'}{d'^2}\int_o^1\int_o^1\int_o^m\left(\frac{dv}{dy}\right)^2 dx\,dy\,d\tau - \gamma'\,\frac{V'^2_o}{2}\int_o^1\int_o^m v^2\,dx\,dy = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

If T' of the model is the time corresponding to T of the prototype, the integrals in Equations (28) and (29) are equal in value, and, also, $\alpha = \alpha'$, $\beta = \beta'$, and $\gamma = \gamma'$, approximately, since vertical velocities are neglected. Hence, the following relations are valid:

$$\frac{g\,s\,d}{g'\,s'\,d'} = \frac{V_o^2\,T\,\nu\,d'^2}{V'^2_o\,T'\,\nu'\,d^2} = \frac{V_o^2}{V'^2_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

If T and T' are corresponding times, it can be shown that $T = \frac{L_o}{V_o}$, $T' = \frac{L'_o}{V'_o}$, so that instead of Equations (30), the following may be written:

$$\frac{g\,s\,d}{g'\,s'\,d'} = \frac{\nu\,V_o\,L_o\,d'^2}{\nu'\,V'_o\,L'_o\,d^2} = \frac{V_o^2}{V'^2_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

and, therefore,

$$\frac{\nu\,L_o}{g^{0.5}\,s^{0.5}\,d^{2.5}} = \frac{\nu'\,L'_o}{g'^{0.5}\,s'^{0.5}\,d'^{0.5}} = \kappa \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

in which, κ is a constant. This agrees in form with the model law obtained by Messrs. O'Brien and Cherno, except that, in Equation (32), ν represents the ratio of $\mu_1 + \mu_2$ to $\rho_1 + \rho_2$, and s the ratio of $\rho_1 - \rho_2$ to $\rho_1 + \rho_2$; whereas, in the law as stated by them, ν is the kinematic viscosity of the salt water and s the defference between the specific gravity of the salt and fresh water.

The foregoing derivation of the model law has been presented primarily because of its theoretical significance. In experiments in which the differences, $\rho_1 - \rho_2$ and $\mu_1 - \mu_2$, are small, as is the case here, and in which the uncertainties in the observation of the position of the salt-water wave are relatively large, it makes little difference whether the model law developed by the authors, or the law developed in this discussion, is used. Although the authors do not discuss their errors of observations, it seems probable that these were large enough to make it impossible to distinguish which model law agrees better with the experimental data.

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DISCUSSIONS

METEOROLOGICAL DATA
PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY MESSRS. CHARLES W. SHERMAN, J. C. STEVENS, THORNDIKE SAVILLE, A. L. SONDEREGGER, AND EDWARD HYATT.

CHARLES W. SHERMAN,[21] M. AM. SOC. C. E. (by letter).[21a]—While recognizing the large amount of study which the Committee has given to the subject, the writer feels that some of the criticisms of the Weather Bureau, expressed or implied, are more severe than the facts justify; or that, in some cases, criticisms should be directed, not at the Bureau, but at the parties who have most to gain from a wider distribution of stations, and an extension of the observations. It may be claimed that the Weather Bureau has gone as far as the necessities require in establishing regular stations with paid observers; and there is room for difference of opinion as to the extent to which it is justified in going in securing co-operative observers. State or municipal officers, power companies, and others to whom meteorological data are of significance, can readily establish as many stations as they wish, and can be assured of co-operation from the Weather Bureau, sometimes to the extent of furnishing the instruments, and always with instructions for setting instruments, taking observations, and furnishing forms for records. Cases frequently arise, of course, in which meteorological data for a given region would be of value, but do not exist. No one can be blamed for not beginning the accumulation of records before their value became obvious, or in an unsettled country.

The Committee's Recommendation 3, that co-operative stations be inspected, is of great importance. The writer would go further than the Committee and suggest that such inspections be made frequently—not less often than once in five years—and by a qualified employee of the Bureau. That through lack of care or of intelligent supervision records may become worthless or misleading, may be illustrated by a few examples.

NOTE.—This Progress Report of the Special Committee on Meteorological Data was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in January, 1933, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: April, 1933, by Messrs. C. S. Jarvis, C. F. Marvin, and Ivan E. Houk; and May, 1933, by C. E. Grunsky, Past-President, Am. Soc. C. E.

[21] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[21a] Received by the Secretary May 19, 1933.

The late Desmond FitzGerald, Past-President, Am. Soc. C. E., once told the writer that soon after he became Superintendent of the Western Division of the Boston, Mass., Water-Works, he found, in a storehouse at Lake Cochituate, a rain gauge to the rim of which a piece of tin had been soldered, thus increasing the diameter of the gauge. He thought the circumstantial evidence proved that this gauge had been used for several years, but could not find anything definite. His opinion was that the published rainfall records for this station during the Sixties were partly derived from it, and that all the early figures should be rejected.

The City of Springfield, Mass., has maintained a recording rain gauge for many years; from 1891 to 1913, it was on the roof of the old Water Commissioners' Office, and after an interval of about a year it was re-established on the roof of the new City Hall. It is probable that the original collecting funnel was damaged, and that the coppersmith who put up the collector in 1914 made a new one. This had a shallow funnel, a rolled, instead of a sharp, edge and a diameter of 8 in., instead of $7\frac{1}{2}$ in., the size of the original collector. This condition was not discovered until 1919, so that all the records obtained during the interval were too great.

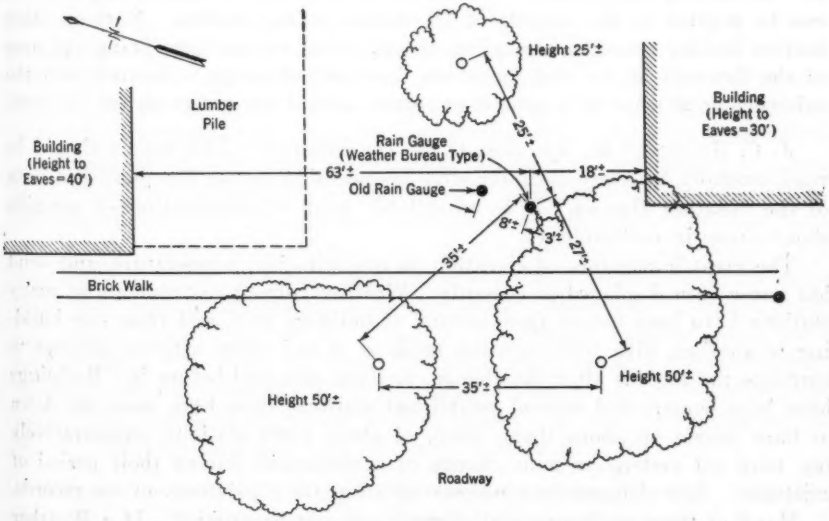

FIG. 10.—EXPOSURE OF RAIN-GAUGE AT U. S. ARMORY, SPRINGFIELD, MASS.

The official rainfall records at Springfield are those obtained at the U. S. Armory, a co-operating station. Although located more than half a mile distant from, and 100 ft higher than, the gauge at City Hall, a study of the Armory records was made, to see if they gave any indication of deficient catch of the roof gauge at City Hall. Considering only storms in which there was a period of intense precipitation, it was found that the catch recorded by the automatic gauge was less than that at the Armory by 4.2%

for eighteen storms in 1892–97; less by 1.7% for twelve storms in 1898–1903; greater by 1.6% for sixteen storms in 1904–08; greater by 2.5% for six storms in 1909–13; and (after adjustment for excessive size of collecting funnel), 11.1% greater for thirteen storms in 1915–18. In other words, instead of the roof gauge showing a smaller catch than the ground gauge (as it did prior to 1903), it showed in 1919 a materially larger catch, and the difference was increasing with the passage of time. The probable reason for this is shown by reference to the plan showing the surroundings of the Armory gauge (Fig. 10). It is likely that the trees which, in 1919, almost overhung the gauge, were too small to have much effect on the records prior to 1900.

With regard to the magnitude and significance of the errors resulting from the location of stations on the roofs of buildings, the writer does not believe that any definite factor can be applied for the adjustment of observations. Confining the discussion to precipitation records alone, the writer[22] found ratios between the catch of a gauge on the ground and one elevated 25 ft above the ground, varying from 0.95 to 1.56; these applied to all the storms containing high intensity rainfalls in the 50-year record at Chestnut Hill Reservoir, near Boston. He is convinced, therefore, that no single factor can be applied to the records of all storms at any station. Various other factors besides elevation also affect the relation, among them being the area of the flat part of the roof on which the elevated gauge is located, and the existence or absence of a parapet or screen around the gauge on the the roof.

J. C. STEVENS,[23] M. AM. SOC. C. E. (by letter).[23a]—This report should be read carefully by every engineer who has occasion to use the published data of the Weather Bureau. It is a veritable mine of information—a genuine short course in meteorology.

The great importance of elevation on precipitation, temperature, and wind has never been displayed so cogently. Weather Bureau instruments at many stations have been moved from ground to building roof, and from one building to another, with little mention made of it and often with no attempt to correlate the records after the change, to those obtained before it. Buildings have been constructed around established stations, trees have been cut down or have grown up about them, until, of about 6 800 stations, comparatively few have not undergone some change of environment during their period of existence. Any changes must necessarily affect the consistency of the records.

Many of these environmental changes can not be avoided. If a Weather Bureau station stands in the way of progress it must be moved. In some cases it might have been possible to have secured comparative data under the old and new conditions, and thereby obtained factors by which the record for the two periods could be made more comparable. Perhaps this has been done in some cases; in others, the change has been gradual, and no such correction factors could have been obtained. In a city every building located within a

[22] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 952.
[23] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.
[23a] Received by the Secretary May 22, 1933.

fairly large radius of the Weather Bureau station has undoubtedly had its influence on wind and temperature, and, hence, on the catch in the rain gauge.

The writer believes that a fewer number of well equipped and better supervised stations properly distributed would give far more useful data than the present large list of co-operative stations supervised almost wholly by correspondence. The results from these co-operative stations have entered most persistently and extensively into the data forming the basis of large developments for irrigation, power, and municipal water supplies. The West is studded with monuments to the inconsistency of much of those data as well as to the stupidity—or, perhaps, credulity—of those who used them.

Precipitation data are those most widely used by engineers, and one engineer's use of them is the equivalent of their use by entire communities. By force of circumstances, perhaps, precipitation data are most abundant where populations are the greatest, because co-operative observers are available; but most water supplies come from large and often inaccessible areas where only a few people live. These are the areas from which precipitation data are the most valuable.

The development of reliable long-time recording instruments is the only practicable solution to this problem of obtaining climatological data in inaccessible areas. For instance, an instrument that would make a record of temperature and precipitation for an 8 or 9-month period with one setting, would fill a long-felt want. It could be installed in a proper environment that would probably never change and would only need to be visited for setting and removing the record at the beginning and end of each winter season.

In conclusion, the writer wishes to pay a tribute to the officials and staff of the Weather Bureau for a great amount of excellent work they have done. Without the data the Bureau has collected engineers would have been at a colossal disadvantage, particularly in the development of the West. It is impossible to count or define the numberless times and manners in which these data have influenced the lives and social institutions in this country. The task of the engineer is to do more than criticize—he must build better than before. The work of this Committee must result in constructively pointing out the weak places in the present system and offering practical suggestions for the betterment of the service. In this, it is certain, the Weather Bureau itself will lend its hearty co-operation.

THORNDIKE SAVILLE,[24] M. AM. SOC. C. E. (by letter).[24a]—The Committee has made a painstaking analysis of the scope and functions of the U. S. Weather Bureau, with particular reference to the utilization of its services by engineers. Many of the suggestions made for improving these services should be helpful in connection with the re-organization of Federal agencies stated to be one of the imminent objectives of the present Administration.

The writer has had a long and intimate contact with Weather Bureau activities, both as an engineer engaged in hydrological studies, and as an administrator of a State agency co-operating extensively with the Weather

[24] Prof. of Hydr. and San. Eng., New York Univ., New York, N. Y.
[24a] Received by the Secretary May 26, 1933.

Bureau. Out of this experience has developed a belief that, like most governmental agencies related to Federal, State, or municipal administration, the Weather Bureau over a long period of years has tended to become bureaucratic, and that a re-appraisement of its organization and services would be desirable. However, the Weather Bureau has performed many admirable services, and the report of the Committee would have done well to include some measure of justifiable praise, as well as pertinent criticism.

Federal agencies such as the Weather Bureau should be essentially service, and not research, organizations. It may well have a research branch to undertake studies of a character unsuited to individual or collective effort outside the Goverment, and certainly a significant proportion of its personnel should be sufficiently interested in and qualified for research to utilize the facilities available in their routine duties to advance the science of meteorology and its applications. Basically, however, the Weather Bureau is charged by law with two definite service functions. The first is the collection and publication of fundamental meteorological data. The second is the utilization of such data for the purpose of forecasting weather conditions in the interests of agriculture, commerce, navigation, and the public welfare. Such research as the Weather Bureau may undertake officially should be devoted to improvement in these services.

On the whole the collection and publication of fundamental meteorological data have been well performed by the Weather Bureau. Data are published with reasonable promptness and in usable form. Exceptions are the collection and publication of rainfall intensity data and records from recording equipment. The former are only to be found in the Annual Reports of the Chief of the Weather Bureau and in a few station publications. The latter, including traces from thermographs, barographs, etc., are sent from the local offices for filing in Washington, and no copies of the original weekly curves are kept at the local centers. Marked improvement in service would result if these records, or duplicates of them, were kept so that they could be readily inspected and copied if desired.

The writer is entirely out of sympathy with suggestions designed to have the Weather Bureau publish its data for some climatic year instead of the calendar year. No single grouping of months to form a climatic year is applicable to the United States as a whole, and there is no reason why a governmental agency should establish a form of reporting data which is applicable only to one section of the country. The calendar year is internationally used for reporting virtually all natural phenomena (the practice of the U. S. Geological Survey in reporting stream-flow data being a departure of questionable merit), and investigators may re-arrange the data upon any basis to suit their varying needs. Governmental agencies should present facts in reasonably utilizable and standardized form, and not seek to mould those facts for some particular or local use.

Most of the deficiencies in the services rendered by the Weather Bureau are due to lack of adequately trained personnel, the deplorable situation in this respect being well emphasized in the Committee's report. The proportion

of scientifically trained men among the full-time technical employees of the Bureau is astonishingly low. The great proportion of the work of collecting and publishing scientific data is necessarily routine, but the judgment and knowledge required to do even such routine work in a fashion conducive to maximum accuracy and with a view toward greatest utility requires a trained technical background.

The writer has had occasion to use long records of river stages kept by the Weather Bureau, and has found that a recognition of accuracy in datums from which the elevations are measured and of accuracy in the recording of observations has been, in important instances, of a very low order. Work of this character is essentially engineering in nature, and this division of the Weather Bureau's activities as well as some others might well be under the direction of a competent engineer.

Evidence of lack of scientific training and appreciation of the widest use of data is contained in several of the special publications of the Weather Bureau. The infrequency of calculations of new normals for temperature, rainfall, etc., has resulted in erroneous inferences being drawn from current data. Such difficulties would be alleviated if these subjects were prosecuted under the direction of more competently trained men.

The recommendation of the Committee for increased co-operation on the part of the Weather Bureau with educational institutions and scientific and technical organizations offers a fertile field for increased usefulness on the part of the Bureau. It is in this way that the Bureau may most effectively stimulate scientific research which will result in improvement of its activities and the usefulness of data collected by it. The writer knows of numerous occasions where competent scientific investigators attached to reputable technical institutions or organizations, and having adequate facilities for extensive research projects, have been prevented from engaging in such projects solely because of lack of equipment. If, after approving a co-operative research project, the Weather Bureau was in a position to loan standard equipment for a reasonable period of time, many valuable additions to meteorological science would result from a very moderate expenditure.

In his experience with Weather Bureau officials the writer has received uniformly courteous treatment, and has always found throughout the service a desire to supply information and assistance within the limitations of the existing organization. It is true that to provide some of the improvements suggested herein would require increased expenditures. Such increases, however, would be relatively slight when the expenditures of the Bureau as a whole are considered.

To promote a well conceived program of service reasonably balanced as to objectives, the writer ventures the suggestion that an advisory council to the Weather Bureau would be helpful. Such a council could well include scientists from groups representing meteorology, physics, mathematics, engineering, aeronautics, agriculture, commerce, etc. Its personnel would presumably be glad to serve without pay provided their expenses were allowed for one or two meetings a year with the Weather Bureau officials in Washing-

ton. Such a group would be helpful in assisting the Bureau to formulate improved procedures, and as a medium of official contact between the Bureau and the major public interests which it is the function of the Bureau to serve.

A. L. SONDEREGGER,[25] M. AM. SOC. C. E. (by letter).[25a]—Meteorological data as collected and published by the Weather Bureau are of general interest. However, different climatic conditions in various parts of the country demand that emphasis be laid on different phases of meteorological observations. The Committee is representative of the engineers of the semi-arid and arid West, where phenomena connected with water supply and water conservation assume supreme importance. An effort by the Chief of the Weather Bureau to personally contact, in the various regions, those who are directly interested would unquestionably tend to extend observations and research along lines which are of greatest local importance, and thus contribute toward maintaining the practical usefulness and efficiency of the service.

In Southern California, partly as a result of local demand, the following services are now available: (1) Standard information of first-class stations; (2) regular daily forecast, except Sundays and holidays; (3) storm signal service; (4) frost service; (5) aviation service (weather reports and forecasts for flying); and (6) special monthly report on precipitation of about 100 regular public and private stations.[26]

The frost service is a special local service maintained for the citrus growers. The special "rainfall data" service is also a local institution, necessitated by the installation of numerous rain-gauges by local interests.

In addition, since about 1919, the Los Angeles Chamber of Commerce has maintained a Department of Meteorology and Aeronautics in charge of a trained meteorologist, at which climatic data are analyzed and the results made available for public use.[27]

Relative to Conclusion (10) of the report, private precipitation stations as a rule are not inspected by the personnel of a regular U. S. Weather Bureau station, such inspection being no part of the routine. This lack of attention may result in the publication of erroneous data and does not tend to develop, or maintain, the proper morale of private observers, the majority of whom contribute their services without any compensation.

Referring to Conclusion (11), while unquestionably the records collected at the regular Weather Bureau stations are complete and reliable, they may not be the most useful to the public if location and exposure are not indicative of actual climatic conditions. Although the engineer may be aware of these facts, he may have to use the data, not having the time or information (nor the inclination, for that matter) required to make necessary corrections. Thus are introduced errors in computations and deductions which might be avoided.

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[25] Cons. Engr., Los Angeles, Calif.

[25a] Received by the Secretary May 31, 1933.

[26] "Rainfall Data for Los Angeles, Orange, Riverside, and San Bernardino Counties, California."

[27] "The Land of the Beckoning Climate", by Dr. Ford A. Carpenter.

The importance of securing reliable climatic records indicative of local conditions is generally recognized and becomes apparent when it is remembered that investments, amounting to billions of dollars, have been made in the United States for enterprises, the success of which depends, in part, on the reliability of estimates of available water supplies. In many instances, these records covered only short periods of time, adding an uncertainty to investments which might have been avoided had more complete information been available.

This leads to the problem of seasonal and long-range forecasts. There is probably no single factor that would affect more favorably the welfare of the populace at large and tend toward stability in business than a reliable seasonal or long-period forecast.

Long-Range Forecast.—It may reasonably be assumed that climate and water supply are not the result of chance or the whimsical quirks of Nature, but are affected or controlled by basic phenomena which follow more or less stabilized physical laws. Such being the case, it is more logical to assume a cyclic variation of climate than permanent departure from the established order, unless physiographical evidence should tend to prove a gradual or permanent change within relatively recent times. If this is not the case, a repetition of the broad order of things may be expected. Hence, long-range forecast may be based upon historical evidence.

Having further in mind that the earth is spherical and that climatic phenomena at one point may be indicative of, or that they may be the cause of, related phenomena occurring subsequently at some distant point, seasonal forecast might be attempted as a deduction from terrestial facts, involving a co-ordination of meteorological observations made at strategical points.

Last, but not least, forecast might be based upon astronomical evidence as a result of analysis of the basic forces which are responsible for climatic conditions.

A forecast based upon historical evidence should take into consideration available local records, evidence of a physiographic character, such as fluctuations of lakes and the action of streams relative to the terrain, etc., and botanical evidence.

In many localities, precipitation records cover only relatively short periods, affording no satisfactory basis for long-range forecast. In this connection, correlative studies of the long-period records of other localities may be indicative of the range of fluctuation to be expected. However, no compilation of the long-period precipitation records in the United States is available, not to mention foreign records. The research worker will have to collect the "Summaries of Climatological Data" of each of the forty-eight Sections and compile his own tabulations. How to obtain and translate the foreign records is still another problem. Here would seem to be a field of useful study for the Weather Bureau and one that would be appreciated throughout the country.

Relative to terrestrial facts on which to base forecast, the far-reaching inventions in the realm of physical science within the last decades, and the

general progress in the natural sciences, would indicate that efforts at seasonal forecast have a fair chance of success. Apparently, this view prevails with many meteorologists, engineers, and physicists. Reference is made to the work at Scripps Institute of Technology, La Jolla, Calif., relative to forecast of seasonal rainfall on ocean temperature[28]; that of Groissmayr, in Germany[29]; and that of Henry Helm Clayton.[30]

Since time immemorial, astronomers have made seasonal and long-range weather forecasts, presumably with varying degrees of success. The subject is as acute as ever, and the prospect of a solution is probably better than ever. Of Federal agencies, the Smithsonian Institution is stated to be engaged in research which has a bearing on climatic cycles and long-range forecast. Reports on conferences are available between representatives of various institutions, including the Smithsonian Institution and the Weather Bureau, at which the existence of cycles of various kinds has been the chief theme, with prediction as a goal.[31] It appears that thus far the Weather Bureau has refused to incorporate research along the lines of long-time weather forecast in its activity.

Generally speaking, it is apparent that the resources at the disposal of private institutions and individuals cannot measure up with those of the Federal Government which, in addition to larger funds, has at its command the services of the Navy and Merchant Marine, exercises control over radio broadcasting, and has an organized Weather Bureau which covers the American Continent, as well as the insular possessions in both the Atlantic and Pacific Oceans. Well-directed research on the part of the Government relative to the problem of seasonal weather forecast, therefore, should be far more promising of success.

Federal Bureaus engaged in other lines of research, such as plant industry, general agriculture, and irrigation, have achieved notable results which have been of immeasurable benefit to the country at large. These Bureaus are occupied in original research and generally have taken the lead in their line of work. The Weather Bureau might attain a similar position relative to meteorological phenomena.

EDWARD HYATT,[32] M. AM. SOC. C. E. (by letter).[32a]—The report of the Committee indicates extensive study and careful analysis of the service now being performed by the United States Weather Bureau, and the personnel of the Committee is such as to warrant thoughtful consideration of its conclusions and recommendations. It is hoped that responsible officials of the Federal Government will give thorough consideration to the report, and it is

[28] Articles by Dr. George F. McEwen, Prof. of Physical Oceanography, Scripps Institute of Oceanography, La Jolla, Calif., and "Problems of Seasonal Weather Forecasting", by Dr. Charles F. Brooks, Prof. of Meteorology and Climatology, Clark Univ., *Professional Engineer*, November, 1927, pp. 9–11.

[29] Fred Groissmayr, Meteorologist, Passau, Germany: (*a*) "Correlation Between Argentina Pressure and Temperature in the United States Six Months Later", *Monthly Weather Review*, July, 1926, p. 299; (*b*) "Correlation Studies of Temperature in Eastern United States", *Loc. cit.*, January, 1929, p. 20.

[30] "World Weather", by H. H. Clayton.

[31] Repts. on the Conferences on Cycles, pub. by Carnegie Inst. of Washington, 1929.

[32] State Engr. of California, Sacramento, Calif.

[32a] Received by the Secretary June 20, 1933.

anticipated that time will show its beneficial influence. The Committee should be congratulated on the progress which it has made and its work should be continued.

That there has been some dissatisfaction with the work of the Weather Bureau is evidenced by a resolution adopted[33] by the Association of Western State Engineers at its Fourth Annual Conference in Sacramento, Calif., on October 30, 1931. This is an association of those officials in the seventeen arid and semi-arid States of the West who are charged with the responsibility of administering the laws governing the appropriation, distribution, or control of the water resources of those States. One of the subjects included in the agenda of this conference was "Improvement of Meteorological Service," and the resolution adopted was to the effect that (*a*) Weather Bureau stations located on high steam-heated buildings did not represent either the temperatures on the ground surface where crops are grown, or true rainfall conditions; (*b*) records of voluntary observers are published without adequate knowledge either of the type of equipment used or its exposure; (*c*) observation stations should have a wider distribution among rural areas in such manner as to represent typical physical characteristics; (*d*) the Weather Bureau should increase its original research work, and, particularly, there should be a persistent and more extensive study of the possibility of long-time forecasting; and (*e*) the personnel of the U. S. Weather Bureau is not of a sufficiently high grade, scientific character.

On May 11, 1932, the President of the Association of Western State Engineers appointed a committee consisting of George W. Malone, Assoc. M. Am. Soc. C. E., S. T. Harding, M. Am. Soc. C. E., and the writer, to contact and co-operate with the Special Committee of the Society and to make any independent studies which seemed advisable. This Committee has not yet (1933) completed its report, but the preliminary conferences which have been held indicate substantial agreement with the Committee of the Society.

It is believed that the Weather Bureau has not done as much in the way of original scientific research as might have been expected from a great scientific bureau of the Government, nor as much as has been done and is being done by other bureaus. Private financing cannot be depended upon for such research. With proper sympathetic understanding between the Bureau and those having greatest need for its services, and for the results of such research work as it may properly undertake, a way should be found to secure increased appropriations for the work if these are necessary. It is not sufficient that the Bureau by contact with meteorological leaders should keep abreast of the scientific progress of meteorology throughout the world, or that its experts should participate in oral and written discussions with other experts. Original research and leadership are desirable.

The great service which has been and is being performed by the Weather Bureau is fully appreciated and the fact that the report of the Committee is of a critical nature should be attributed, not to any lack of appreciation on the part of the Committee, or of engineers in general, but to the original

[33] *Proceedings*, Fourth Annual Conference of the Assoc. of Western States Engrs., 1931, p. 239.

instructions of the Committee to "give thought as to how the United States Weather Bureau could be made of greater service to engineers." Obviously, this purpose could only be served by citing what were, in the judgment of the Committee, weaknesses in the existing service of the Bureau.

That both temperature and precipitation are profoundly influenced by elevation above ground surface appears to be commonly recognized among experts, evidence of which fact is presented by the Committee. Doubtless, few observing people have failed to note heavy frost and frozen puddles on occasions when the Weather Bureau report for the local station situated on some tall office building showed a minimum daily temperature in excess of 32°F.

Civil engineers at least are interested most in conditions at or near ground level, and if the records of stations on tall buildings do not truly reflect such conditions they will desire some correction factor with which to relate the two. The Weather Bureau should be best prepared to supply this factor and obviously it is not desirable for each individual having need of the data to exercise his own best judgment in an attempt to improve these original facts according to his ideas of what corrections should, or should not, be applied. Either the records at these elevated stations are correctly representative of ground-level conditions or they are not. If they are, the matter is properly disposed of by so showing. If they are not, the civil engineer desires to have the best practicable correction factor and it should be estimated once and for all.

Some years ago the writer felt the need of some ready means by which to establish current rainfall conditions as compared with normal, and cumulative mass diagrams were developed for California, in the form shown in Fig. 11.

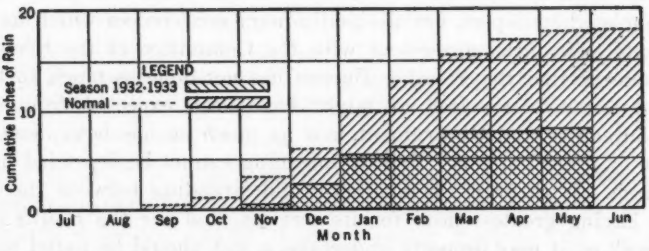

FIG. 11.—MASS DIAGRAM OF CUMULATIVE MONTHLY RAINFALL FOR SACRAMENTO, CALIF. (ELEVATION, 71 FEET; AVERAGE ANNUAL PRECIPITATION, 17.95 INCHES.)

Fifteen typical precipitation stations were selected—five in Northern California, five in Central California, and five in Southern California—well distributed as to geographical location and elevation. The normal cumulative percentage of total seasonal precipitation at each station was computed for the end of each month and so represented upon a diagram by hachuring that the current precipitation might be shown over it in full color, thereby making it possible at a glance to compare current precipitation with the normal for the station. This information for the fifteen stations is included on a single plate, 11 by 22 in., and included therewith is a statement as to

the elevation of each station and its normal precipitation in inches. The plate has been found most useful and there is a considerable demand for it among engineers and agencies who learn of its exitsence. If the Weather Bureau were to prepare such a diagram showing weekly instead of monthly cumulative totals, there is little doubt that the innovation would meet with general approval.

Development of various kinds during recent years in the mountainous areas as a result of settlement, construction of hydro-electric power and reservoir projects, etc., has opened the way for co-operative stations where none now exist, and records at such stations are highly desirable. It is felt that this co-operation could probably be arranged at little or no cost. The writer's office has received very substantial contribution to its snow survey work in recent years in the way of free co-operation by the Weather Bureau, the National Parks Service, and the U. S. Forest Service, and by cities, irrigation districts, power companies, etc., operating in the mountain areas. The functions of the Weather Bureau might well be extended to include active direction of this work.

By reason of the nature of its organization and method of functioning, with the responsible head far removed from intimate contact with the majority of beneficiaries of the service which it performs, the U. S. Weather Bureau is in a position to profit by wide discussion of the work of this Committee. It is hoped that this report will be accepted as merely a progress report of the Committee and that its work will be continued.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EVAPORATION FROM WATER SURFACES
A SYMPOSIUM

Discussion

BY MESSRS. S. T. HARDING, L. STANDISH HALL, AND ADOLPHO SANTOS, JR.

S. T. HARDING,[28] M. AM. SOC. C. E. (by letter).[28a]—This Symposium brings together much information that will be helpful to engineers having to do with work in which evaporation from water surfaces may be involved. The presentation of available records obtained from various types of pans and their relationship to the evaporation from large water surfaces furnishes a guide for the interpretation of existing data. The recommendations regarding standard pans for use in future observations will enable many of the variations of past observations to be avoided

Measurement of evaporation presents many difficulties. This is particularly true of floating pans where, if the pan is placed far enough from the shore to secure conditions of exposure that are representative of the larger water surface, it is difficult to secure freedom from wave splashing without in turn affecting the exposure. Unforeseen conditions may arise which are difficult to overcome. In the writer's floating-pan records on Buena Vista Lake, which are quoted in the Symposium, inconsistencies occurred in the observations for the earlier months which could not be explained until the observer chanced to see pelicans using the pan as a bath-tub. It was necessary to place a coarse wire screen above the pan to prevent this loss of water.

While the material in the Symposium covers the effect of type and size of pan thoroughly, there are other factors which affect the relationship of the evaporation from pans and from large water surfaces which may make as much difference as variations in the size of pan. This is illustrated by the two following cases.

NOTE.—This Symposium on Evaporation from Water Surfaces was published in February, 1933, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings* as follows: May, 1933, by Messrs. Ralph R. Randell, C. E. Grunsky, and Charles H. Lee.

[28] Prof., Irrig., Univ. of California, Berkeley, Calif.

[28a] Received by the Secretary May 9, 1933.

In describing[20] the work of the U. S. Geological Survey in the Escalante Valley, Utah, Mr. W. N. White gives the results of observations of pans 20 in. and 12 ft in diameter. Two of the smaller pans were used. In one set of observations the two smaller pans floated in the 12-ft pan, being set in the direction of the prevailing wind; in the other set, they were set in the ground adjacent to the 12-ft tank in the same directions. From May to August, 1927, the 12-ft pans had an evaporation of 23.34 in., the windward floating pan, 28.44 in., and the leeward floating pan, 24.49 in. For a period in August and September, 1927, the 12-ft pan had an evaporation of 7.89 in., the small sunken land pan on the windward side, 10.15 in., and the similar land pan on the leeward side, 8.76 in. In the case of the small floating pans the evaporation on the windward pan was 16% greater than that for the leeward pan; for the two sunken land pans, the evaporation on the windward pan exceeded that on the leeward pan by the same average percentage.

In Table 8 of Mr. Follansbee's paper the mean evaporation from April to September from a reservoir surface at Fallon, Nev., is given as 44.50 in., based on records from 1908 to 1930, with a pan having a coefficient of 0.94. This is the pan at the Fallon Experiment Station of the U. S. Department of Agriculture. Observations were begun in 1908 and were continued to 1920, when the station was closed until 1922, when it was re-opened. The evaporation observations have been continued to date(1933). Recently, while making some studies of evaporation in this area, the writer noted that the results since 1922 showed smaller amounts of evaporation than those for the earlier period. For the months from April to September, for the 12 years, from 1908 to 1919, the evaporation from the pan varied from 47.13 to 55.04 in., with a mean of 51.64 in. For the 10 years, from 1923 to 1932, the evaporation for the same months varied from 36.47 to 44.97 in., with a mean of 39.45 in. The mean for the earlier period exceeded that for the latter period by 30%; the evaporation in the maximum year in the later period was only 95% of the evaporation in the minimum year of the earlier period.

Mr. E. W. Knight, in charge of the Experiment Station, states that when the station was re-opened in 1922, the evaporation pan was moved from a small hill in the farmed area of the station to a level area near on irrigated field.. Records of wind movement obtained adjacent to the pan show an average reduction in wind velocity of about 25% at the site used since 1922. There has been little difference in mean temperature for these months. While there are no records on large water areas in this vicinity covering both parts of this period, there is no reason to expect that records would have shown any essential differences in actual evaporation from such large water surfaces in the earlier and later years.

In the observations in Escalante Valley, a water surface 12 ft in diameter set between two small pans resulted in differences in evaporation of 16% due to differences in humidity or other factors. In the observations at Fallon moving the evaporation pan from a small hill about 10 ft to a level area near

[20] *Water Supply Paper 659-A*, U. S. Geological Survey.

an irrigated field, but still within the same 160-acre tract and subject to the same general climatic conditions, resulted in a 30% difference in evaporation. These results indicate that the conditions of exposure may be a larger factor in the relationship between the evaporation from pans and large water surfaces, than the factors due to the size of the pan.

L. STANDISH HALL,[30] ASSOC. M. AM. SOC. C. E. (by letter).[30c]—A great volume of data have been gathered for this Symposium. In a strict sense, the evaporation from reservoir surfaces is still a matter of question. The coefficients for some of the types of pans in use show a wide range, and although reasonable values have been determined for reducing annual pan evaporation to equivalent reservoir-surface evaporation, little is known of the ratio to be applied in individual months.

The engineers of the East Bay Municipal Utility District (and its predecessor, the East Bay Water Company) have made comparisons of evaporation from different types of pans since 1921. In that year four pans were installed at the San Pablo Reservoir, near Oakland, Calif. All the pans were 3 ft square and 1.5 ft deep, two being Type 2 pans and two being Type 4 pans. One land and one floating pan were located near the dam, and another pair near the outlet tower about 7 000 ft up stream from the dam. The floating pans were anchored near the shore, and the land pans were kept about 10 to 20 ft in elevation above the reservoir level. Rain-gauges were maintained near the land pans. The pans were moved at various times according to the elevation of the water surface in the reservoir, but were kept in the same relative positions. Due to an error in attaching the floats to the reservoir pans, the water level in the pans stood between $\frac{3}{4}$ in. and 1 in. higher than the outside water surface.

In 1930, a new evaporation station was installed at this reservoir near the dam. The land station consisted of one Type 2 pan and one Type 3 pan, with anemometer, maximum and minimum air thermometer, and standard rain-gauge. This station was located on top of the dam. The reservoir station, consisting of one Type 4 floating pan and a standard rain-gauge, was located at the same place as the old floating pan near the dam.

At Upper San Leandro Reservoir, 12 miles southeast of San Pablo Reservoir, a land pan (Type 3) and a floating pan (Type 4) were maintained near the dam from 1925 to 1929. The record for this period is fragmentary. In 1930, a new station was installed near the outlet tower. This station was a duplicate of that placed at San Pablo in the same year.

At Pardee Reservoir, near Valley Springs, Calif., about 80 miles east of Oakland, an evaporation station was installed in 1930. The equipment was similar to that at San Pablo Reservoir, with the exception that a Class A Weather Bureau pan was included with the pans at the land station.

The available data, reduced to monthly averages for each of the aforementioned stations, is included in Table 13.

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[30] Chf. Hydrographer, East Bay Municipal Utility Dist., Oakland, Calif.

[30c] Received by the Secretary June 28, 1933.

A comparison of the ratio of evaporation for the various types of pans shows a marked difference between the relative evaporation from land pans and floating pans between the winter and summer months. Even as between two land pans of different types the ratio is not constant throughout the year. Evaporation records from stations other than those at the District's reservoirs, although they show different values of the ratio of evaporation from pans of the several types, do not indicate the marked difference between summer and winter evaporation. In many localities the maintenance of evaporation stations during the winter months is impossible because of freezing temperatures, so that a complete annual comparison is not available.

On the basis of these data, the writer cannot agree with the conclusion of the Sub-Committee that the Class A Weather Bureau pan (Type 1) is preferable for an evaporation station "operated primarily to determine the evaporation from large surfaces in the immediate neighborhood." The test at Fort Collins, Colo., on the 85-ft reservoir, shows that the pan floating in the reservoir followed the reservoir evaporation month by month more closely than the land pans. The Class A pan showed the greatest range of coefficient. The evaporation varies with wind, temperature, and humidity, and in a reservoir the temperature of the water may differ greatly from that in a small land pan. With regard to the East Bay District's reservoirs, the results obtained from land pans and water pans can be easily explained. The reservoirs are relatively deep and, in the winter months, the water temperature in the reservoir is greater than in the land pans, resulting in relatively greater evaporation. In the summer months the converse is true, with the result of relatively less evaporation.

Judging from the results at some of the other stations, where comparisons of land pans and water pans are available, similar conditions did not prevail. One difficulty in the comparison of land pans and floating pans is that wind conditions may not be the same at both pans, even if the pans are relatively close to each other. A wide range in evaporation can be seen in the results at San Pablo Reservoir since 1930 (Table 13(*b*)). The land pans on top of the dam are exposed to the full sweep of the prevailing winds. The floating pan is sheltered from the wind by the dam, and has materially less evaporation. Another difficulty in the comparison of floating-pan records is that a large majority of the floating pans are not completely immersed, but have the water level in the pan at a higher level than in the surrounding reservoir. The floating pan near the dam at San Pablo Reservoir was not completely immersed from 1921 to 1929, but after 1930 the water level in a pan at the same location was about $\frac{1}{4}$ in. below the water level in the reservoir. The mean annual pan evaporation for the first period was 47.8 in., and for the second period, 40.9 in. Some of this difference may have been due to climatic variation and some to the inflow of cold water from Pardee Reservoir during the latter period. It does not seem probable that the entire difference was due to pan immersion.

Buried pans may be subject to varying temperature control. When buried in clay or adobe soil, the shrinkage of the earth around the pan during the

TABLE 13.—COMPARISON OF EVAPORATION PANS, EAST BAY MUNICIPAL UTILITY DISTRICT, OAKLAND, CALIFORNIA

Item No.	Month	Years of record	Precipitation, in inches	Temperature of air, in degrees Fahrenheit	Velocity of wind, in miles per hour	Total Evaporation for the Month, in Inches				Ratio of Evaporation			
						Type 4 pan	Type 3 pan	Type 3 pan	Type 4 pan	Column (7) to Column (8)	Column (9) to Column (8)	Column (10) to Column (8)	Column (10) to Column (9)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
(a) SAN PABLO RESERVOIR NEAR OAKLAND, CALIFORNIA, 1921 TO 1929													
Location of pan		...	...	...	...	Near dam	Near dam	Near tower	Near tower	Near dam	...	...	Near tower
Type of pan		...	...	...	...	Type 4	Type 3	Type 3	Type 4	Type 4 / Type 3	...	...	Type 4 / Type 3
Diameter, in feet		...	...	...	...	3*	3*	3*	3*	...	...	...	...
1	January	7	...	...	...	0.96	0.77	0.63	0.84	1.25	...	...	1.33
2	February	7	...	...	...	1.51	1.39	1.21	1.28	1.08	...	...	1.06
3	March	7	...	...	...	2.62	2.36	2.30	2.46	1.11	...	...	1.07
4	April	8	...	...	...	3.88	3.32	3.51	3.82	1.17	...	...	1.09
5	May	8	...	...	...	5.63	5.09	5.50	5.81	1.11	...	...	1.05
6	June	8	...	...	...	6.37	6.12	6.27	6.59	1.04	...	...	1.05
7	July	8	...	...	...	6.82	6.79	7.29	7.04	1.01	...	...	0.97
8	August	8	...	...	...	6.52	6.55	6.45	6.35	1.00	...	...	0.98
9	September	8	...	...	...	5.67	5.58	5.52	6.09	1.01	...	...	1.10
10	October	8	...	...	...	3.94	3.54	3.49	3.99	1.11	...	...	1.14
11	November	8	...	...	...	2.33	1.92	1.83	2.38	1.23	...	...	1.30
12	December	8	...	...	...	1.56	0.93	0.81	1.37	1.68	...	...	1.69
13	Total	...	...	...	...	47.81	44.36	44.81	48.03	1.08	...	...	1.07
(b) SAN PABLO RESERVOIR, NEAR OAKLAND, CALIFORNIA, 1930 TO 1932													
Type of pan		...	...	...	...	...	Type 2	Type 3	Type 4	...	Type 3 / Type 2	Type 4 / Type 2	Type 4 / Type 3
Diameter, in feet		...	...	...	...	...	6	3*	3*	...	...	...	...
14	January	3	4.40	45.3	4.57	...	1.086	1.282	1.181	...	1.18	1.09	0.92
15	February	3	1.96	48.5	3.70	...	1.570	1.856	1.306	...	1.18	0.84	0.70
16	March	3	2.20	52.4	3.50	...	2.967	3.305	2.063	...	1.12	0.69	0.62
17	April	3	1.14	54.5	3.28	...	4.240	4.749	3.412	...	1.12	0.81	0.72
18	May	3	0.78	58.2	3.54	...	5.745	6.327	4.332	...	1.10	0.75	0.68
19	June	3	0.26	61.4	3.86	...	6.976	7.577	5.215	...	1.09	0.75	0.69
20	July	3	0.00	64.2	4.21	...	8.107	8.714	6.162	...	1.08	0.76	0.71
21	August	3	0.00	64.3	4.01	...	7.211	7.944	5.347	...	1.10	0.74	0.67
22	September	3	0.03	63.3	3.62	...	5.385	5.929	4.198	...	1.10	0.78	0.71
23	October	3	0.68	59.3	3.88	...	4.054	4.608	3.424	...	1.14	0.85	0.74
24	November	3	2.19	54.1	4.24	...	2.402	2.832	2.460	...	1.18	1.02	0.87
25	December	3	4.41	44.9	4.57	...	1.155	1.431	1.767	...	1.24	1.53	1.23
26	Total	...	18.05	...	...	...	50.898	56.554	40.867	...	...	...	...
27	Mean	...	...	55.9	3.92	...	...	...	...	...	1.11	0.80	0.72
(c) UPPER SAN LEANDRO RESERVOIR, NEAR OAKLAND, CALIFORNIA, 1930 TO 1932													
28	January	3	5.14	42.5	1.73	...	0.792	0.880	1.183	...	1.11	1.49	1.34
29	February	3	2.53	45.8	1.76	...	1.199	1.297	1.368	...	1.08	1.14	1.05
30	March	3	2.69	51.2	1.72	...	2.237	2.294	2.312	...	1.02	1.03	1.01
31	April	3	0.97	52.9	1.69	...	3.493	3.629	3.943	...	1.04	1.13	1.09
32	May	3	1.14	57.6	2.15	...	4.951	5.113	5.172	...	1.03	1.04	1.01
33	June	3	0.35	60.7	2.26	...	5.870	6.101	6.084	...	1.04	1.04	1.00
34	July	3	0.00†	63.7	2.24	...	7.128	7.396	7.396	...	1.04	1.04	1.00
35	August	3	0.00	63.3	2.14	...	6.114	6.387	6.446	...	1.05	1.06	1.01
36	September	3	0.11	61.8	1.81	...	4.504	4.713	5.162	...	1.05	1.15	1.10
37	October	3	0.86	57.9	1.67	...	3.261	3.441	4.125	...	1.06	1.27	1.20
38	November	3	2 62	51.4	1.55	...	1.736	1.923	2.623	...	1.11	1.51	1.36
39	December	3	5.64	41.7	1.56	...	0.846	0.943	1.777	...	1.11	2.10	1.88
40	Total	...	22.05	...	...	...	42.131	44.117	47.555	...	...	...	...
41	Mean	...	...	54.2	1.86	...	...	...	...	...	1.05	1.13	1.08

* Square. † Trace.

TABLE 13—(*Continued*)

Item No.	Month	Years of record	Precipitation, in inches	Temperature of air, in degrees Fahrenheit	Velocity of wind, in miles per hour	Total Evaporation for the Month, in Inches				Ratio of Evaporation			
						Type 4 pan	Type 3 pan	Type 3 pan	Type 4 pan	Column (7) to Column (8)	Column (9) to Column (8)	Column (10) to Column (8)	Column (10) to Column (9)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
(*d*) Pardee Reservoir, Near Valley Springs, California, 1930 to 1932													
Type of pan						Type 1	Type 2	Type 3	Type 4	Type 1 / Type 2	Type 3 / Type 2	Type 4 / Type 2	Type 4 / Type 3
Diameter, in feet						4	6	3*	3*				
42..	January	3	3.65	44.2	1.35	0.674	0.522	0.636	1.452	1.29	1.22	2.78	2.28
43..	February	3	2.83	48.7	1.10	1.574	0.983	1.173	1.420	1.59	1.18	1.43	1.21
44..	March	3	1.71	55.1	0.97	3.014	2.124	2.281	2.474	1.42	1.07	1.16	1.08
45..	April	3	1.13	60.2	0.75	4.756	3.720	3.870	4.319	1.28	1.04	1.16	1.12
46..	May	3	1.01	66.0	1.12	7.811	6.079	6.323	6.198	1.29	1.04	1.02	0.98
47..	June	3	0.14	73.1	1.37	10.362	8.078	8.558	7.610	1.28	1.06	0.94	0.89
48..	July	3	0.00	80.7	1.37	13.076	10.210	10.898	10.215	1.28	1.07	1.00	0.94
49..	August	3	0.05	78.3	1.20	11.219	9.019	9.625	9.706	1.24	1.07	1.08	1.01
50..	September	3	0.05	72.1	1.13	7.757	6.077	6.674	7.204	1.28	1.10	1.18	1.08
51..	October	3	0.39	64.5	1.03	4.887	3.563	4.260	5.241	1.37	1.20	1.47	1.23
52..	November	3	1.70	57.0	0.95	2.324	1.663	2.108	3.367	1.40	1.27	2.03	1.60
53..	December	3	3.06	45.6	1.17	0.803	0.575	0.666	2.383	1.40	1.16	4.14	3.57
54..	Total		15.72			68.257	52.613	57.072	61.589				
55..	Mean			62.1	1.13					1.30	1.09	1.17	1.08

* Square.

summer months may result in the sides being exposed to the air. Earth may be removed from around the pan by wind or erosion, exposing the sides below the water level. Pans buried near the water edge of a reservoir or in marshy ground may, in effect, simulate a floating pan. This condition applies to the buried pan at Los Griegos, N. Mex. According to the data[31] originally presented, the ground-water was above the bottom of this pan at all times. Judging from the location of the pans in Fig. 3, a similar condition may have existed relative to the buried pans at the East Park Reservoir, in California.

The Class A Weather Bureau pan, being placed above ground, is not subject to the varying temperature control which may effect buried or floating pans. The results, therefore, may be more consistent although the recorded evaporation does not necessarily follow that from an adjacent reservoir. In cases where the pans cannot be given adequate supervision, this pan has many of the advantages set forth by the Sub-Committee, to recommend it.

To the Sub-Committee's recommendation that two or more pans are highly desirable, the writer would add, that if two pans are used, one should be a land pan and one a floating pan. In windy locations, the use of one pan may be misleading. For example, at the San Pablo Reservoir, the writer has used the evaporation of the floating pan since 1930, without correction, as representing most closely the evaporation from the reservoir surface.

[31] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 984.

Due to the difficulty of maintaining floating pans in a large reservoir, the writer believes that a more accurate comparison of land pans and floating pans could be made by floating the water pan in a tank adjacent to the land pan. For a Type 4 pan, such a tank could be 8 ft in diameter and 3 ft in depth. The tank should be buried in the ground, and an effort made to keep the tank temperature as close as possible to the temperature of the surface water in the reservoir.

The experiments made by the late R. B. Sleight, Assoc. M. Am. Soc. C. E., appear to be very trustworthy. The floating pans, however, cannot be directly compared with the land pans, due to the fact that the pans were not at the same location, as mentioned by Mr. Rohwer. The writer, by comparison of results between land pans and floating pans at Denver, Colo., with the results obtained at Salt Creek Bridge, in California, has revised the coefficients for floating pans obtained by Sleight as shown in Table 14.

TABLE 14.—EVAPORATION FROM CIRCULAR FLOATING PANS (TYPE 5), AND FROM SQUARE FLOATING PANS (TYPE 4).

Type	Diameter, in feet	Area, in square feet	Salt Creek Bridge, California	Denver, Colo., original*	Denver, Colo., revised
5	12.0	113.09	1.00	....	....
5	6.0	28.27	0.95	1.03	0.93
5	4.0	12.56	0.82	0.97	0.87
5	2.0	3.14	0.81	0.89	0.80
5	0.83	0.54	....	0.77	0.69
4	3.0†	9.00	....	0.89	0.80

* From *Proceedings*, Am. Soc. C. E., February, 1933, Table 4, p. 244; from comparison with 12-ft. circular buried land pan not at same location as floating pans.
† Square.

In other instances, in addition to the one just cited, the writer could not agree with the values of the coefficients determined in various parts of the Symposium. The coefficients for the types of pans in most general use, as well as the mean for each type have been assembled in Table 16, and in Table 15 these results are compared with the coefficients determined by Messrs. Sleight, Rohwer, Follansbee, and the Sub-Committee. The coefficient of 0.92 for the 6-ft ground pan was determined from a longer record on the

TABLE 15.—COMPARISON OF COEFFICIENTS TO REDUCE PAN EVAPORATION TO EQUIVALENT RESERVOIR EVAPORATION

Type	Diameter, in feet	Sleight	Rohwer	Follansbee	Sub-Committee	Hall
1	4.0	0.70	0.70§	0.69	0.70	0.69
2	6.0	0.92†	0.94	0.94	....	0.91
3	3.0*	0.79	0.78	....	0.78	0.81
4	3.0*	0.80‡	0.77	0.83	0.80	0.80
5	4.0	0.87‡	0.78	0.83	....	0.88

* Square. †Record for pan 3.0 ft. deep used. ‡ Revised coefficient by Hall. § Recommends 0.69 or 0.70.

pan 3 ft deep at Denver, and is believed to be more representative than the coefficient of 0.94 from the short record on the pan of standard depth.

The comparisons of the Type 3 and Type 4 pans indicate that the evaporation is greater from the floating pan. The results from the East Bay District's measurements are very consistent and show 8% greater evaporation

TABLE 16.—COEFFICIENTS TO REDUCE VARIOUS TYPES OF PAN RECORDS TO RESERVOIR SURFACE EVAPORATION

Item No.	Location	Reservoir with which pans were compared	Years of record	Class A pan (diameter, 4 ft)	Coefficients				
					Buried Pans			Floating Pans	
					(Diameter, 6 ft)	(Diameter, 4 ft)	3 ft square	3 ft square	Diameter, 4 ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Denver, Colo.	12-ft ground pan; Column (7) interpolated	1915–1917	0.70	0.92	0.85	0.79	0.80*	0.87†
2	Fort Collins, Colo.	85-ft reservoir	1926–1928	0.70	.....	.....	0.79	0.77	.....
3	East Park, Calif.	1 800-acre reservoir; Columns (7) and (8) (pans semi-floating, not used in averages)	1930	0.68	.....	0.78	0.75	.....	0.78
4	Milford, Utah	12-ft ground pan	1925–1927	0.67	.....	.....	.....	.....	.....
5	Salton Sea, California.	175 600 acres; indirect method	1909–1910	0.68	.....	.....	.....	.....	0.85‡
6	Lincoln, Nebr.	7.5-ft pan; indirect method	1909–1910	0.66	.....	.....	.....	.....	.....
7	Pardee Reservoir, California.	4-ft, Class A pan	1917–1920	0.68	.....	.....	.....	.....	.....
8	Garnett, Colo.	4-ft, Class A pan; indirect method	1930–1933	.....	0.90§	.....	0.84§	0.76‖	.....
9	Bakersfield, Calif.	10 000-acre lake; Column (9), pan not fully submerged	1927–1930	.....	.....	0.88	.....	.....	.....
10	San Pablo Reservoir, California.	6-ft ground pan	1920	.....	.....	.....	0.82	0.70	.....
11	Upper San Leandro Reservoir, California	6-ft ground pan; indirect method	1930–1933	.....	.....	.....	0.82§	0.75¶	.....
12	Austin, Tex.	4-ft, Class A pan; indirect method	1930–1933	.....	.....	.....	0.87	0.75‖	.....
13	Morena Reservoir, California.	3-ft square ground pan; indirect method	1916–1925	.....	.....	.....	.....	0.88	.....
14	Barrett Reservoir, California.	3-ft square ground pan; indirect method	1928	.....	.....	.....	.....	0.92	.....
15	Lower Otay Reservoir, California.	3-ft square ground pan; indirect method	1928	.....	.....	.....	.....	0.86	.....
16	Los Griegos, N. Mex.	4-ft, Class A pan in saturated soil; indirect method	1928	.....	.....	.....	.....	0.84	.....
17	Nelson Reservoir, Montana.	4-ft, Class A pan; not used in average; indirect method	1926–1928	.....	.....	.....	.....	.....	0.97
18	Oakdale, Calif.	4-ft, Class A pan; not used in average; indirect method	1921–1923	.....	.....	.....	.....	.....	1.01
19	Fall River Mills, Calif.	4-ft, Class A pan; indirect method	1921	.....	.....	.....	.....	.....	1.08
20	Salt Creek Bridge, California.	12-ft circular floating pan	1925–1930	.....	.....	.....	.....	.....	0.91
21	Brooks Reservoir, Alberta, Canada.	4-ft circular ground pan; not used in average; indirect method	1910	.....	.....	.....	.....	.....	0.83
22	Lake Newell, Alberta, Canada.	4-ft circular ground pan; indirect method	1923–1924	.....	.....	.....	.....	.....	1.09
			1923–1924	.....	.....	.....	.....	.....	0.98
23	Means	.....	.....	0.69	0.91	0.86	0.81	0.80	0.88

* 1916 to 1917; indirect method. † 1917; indirect method. ‡ 1910. § Indirect method. ‖ 1930 to 1932. ¶ 1921 to 1929; average of two pans; indirect method.

from the floating pan. Based on these records alone, the coefficients would be: Type 3, 0.85; and Type 4, 0.79. The coefficient for the Type 5 pan appears to be 0.88.

In selecting a pan to determine reservoir evaporation, the writer would reverse the selection recommended by the Sub-Committee and would give preference to the pans in the following order: (*a*) Floating pan, with preference to the U. S. Geological Survey pan; (*b*) buried pan, with preference to the Colorado pan, and then to the 4-ft or 6-ft circular pan; and (*c*) U. S. Weather Bureau Class A pan.

ADOLPHO SANTOS, JR.,[32] JUN. AM. SOC. C. E. (by letter).[32a]—The primary object of the Symposium "has been to find a suitable set of coefficients * * * to the end that evaporation from a large water surface could be expressed in terms of that from the various types of pans in use." In the light of certain researches and experiments conducted on the subject of evaporation from large water surfaces, it does not seem that the appropriateness of the method of attack chosen for obtaining such a suitable set of coefficients is beyond criticism.

The researches and experiments referred to, made principally at the Scripps Institution of Oceanography and at the California Institute of Technology, would appear to indicate that the method adopted by the authors for estimating evaporation from reservoirs on the basis of pan observations yields results of debatable precision. For a proper solution of the problem, the adoption of the point of view and technique originated and advocated by Messrs. G. F. McEwen, N. W. Cummings, I. S. Bowen, B. Richardson, and C. P. Montgomery, is essential. It is to be regretted that not even passing mention is made by the authors of the work of these men[33] who have succeeded in placing the physics of evaporation on a firm foundation.

Messrs. Cummings and Richardson have stated[34] that:

"Evaporation can be determined by the aid of the first law of thermodynamics in such a way that wind velocity need not enter the calculation. Air temperature and humidity enter only as terms in a correction which can have a relatively small average value under typical conditions."

As long as this fact is not fully realized and made use of by engineers, and as long as pans of various types and sizes, under varying conditions of

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[32a] Received by the Secretary June 29, 1933.

[33] "The Ratio of Heat Losses by Conduction and Evaporation from Any Water Surface," by I. S. Bowen, *Physical Review*, Vol 27 (1926), pp. 779–787; "The Measurement of Insolation by Means of a Pan," by B. Richardson and C. P. Montgomery, M. Am. Soc. C. E., *Bulletin No. 68*, National Research Council (1929), pp. 56–61; "Alignment Diagram for 'R' of the Energy-Evaporation Equation", by N. W. Cummings, *Monthly Weather Review*, Vol. 58 (1930), pp. 142–144; "Certain Limitations on the Possible Values of the Ratio of Heat Losses by Convection and by Evaporation at a Water Surface", by N. W. Cummings, *Monthly Weather Review*, Vol. 58 (1930), pp. 144–146; "Results of Evaporation Studies Conducted at the Scripps Institution of Oceanography and the California Institute of Technology", by G. F. McEwen, *Bulletin*, Scripps Inst. of Oceanography, Technical Ser., Vol. 2, No. 11 (1930), pp. 401–415, and "Evaporation as a Function of Insolation", by B. Richardson, *Transactions;* Am. Soc. C. E., Vol. 95 (1931), pp. 996 *et seq.*

[34] "Evaporation from Lakes", by N. W. Cummings and B. Richardson, *Physical Review*, Vol. 30 (1927), pp. 527–534.

exposure, continue to be used, the ratio between evaporation from pans and evaporation from a large open water surface will remain a matter of controversy and dispute, with small gain in factual knowledge of general applicability. As long as empirical methods are used, the "laws," "constants," and "coefficients" resulting from even elaborate experiments must of necessity remain of local and restricted value only.

For example, in water supply problems, it is quite natural to regard variations in wind velocity, temperature, and humidity as the only factors that produce effects on reservoir evaporation sufficiently large to be important. It happens, however, that this very natural view was not supported by an extensive statistical analysis made by Dr. Cummings for the purpose of isolating the effects of the various factors influencing evaporation. In fact, a very weak correlation was indicated between evaporation and humidity and between evaporation and wind.[35] This led Dr. Cummings to attack the problem from the standpoint of the first law of thermodynamics and to formulate the fundamental qualitative principle of evaporation, as follows:

"Any change in wind or humidity is always accompanied by a change in lake temperature which partially neutralizes the direct effect on the evaporation rate to be expected from the original change in atmospheric conditions, if the isolation remains constant."

When the water in a floating pan is kept above the level of submergence, the rate of evaporation from the pan will exceed that of the surrounding water. The writer is not certain, however, that by simply keeping the water in the pan at all times at a lower level than that of the water in which it floats the two rates will become identical. The first question that naturally arises is: How much lower? Is it not conceivable that if the level is low enough the evaporation from the pan will be less than that from the reservoir?

The writer earnestly hopes that the Sub-Committee will adopt the standard procedure and technique advanced by Dr. McEwen, which like most primary standards is difficult to apply. By this means a simpler and more readily applicable method could be developed for determining evaporation from reservoirs with the degree of accuracy sufficient for engineering uses.

The words of Dr. Cummings in discussing Professor Richardson's paper,[36] with slight modification, may be applied to the question being considered:

"It may be some time before the true value [of the procedure based on the first law of thermodynamics] is fully appreciated, because economic pressure has not yet compelled the abandonment of unnecessary complications in estimating evaporation from lakes and reservoirs."

[35] "Relation Between Evaporation and Humidity", by N. W. Cummings, *Bulletin No. 68*, National Research Council (1929), pp. 47–56.

[36] *Transactions*, Am. Soc. C. E., Vol 95 (1931), p. 1017.

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DISCUSSIONS

IMPROVED TYPE OF FLOW METER FOR HYDRAULIC TURBINES

Discussion

BY C. MAXWELL STANLEY, R. C. JOHNSON, AND E. A. DOW

C. MAXWELL STANLEY,[6] JUN. AM. SOC. C. E. (by letter).[6a]—The flow meter for hydraulic turbines, which has been developed by Mr. Winter and his associates, and which is described in his paper, fills a decided need in the field of hydro-electric power development. Hydraulic engineers have long desired and searched for a suitable method for obtaining continuous records of discharge of water through turbines for use as a check on the efficiency of operation. Such a method must be simple, positive, accurate, and economical, if it is to have general application.

Numerous methods are available for the measurement of water. These methods include the weir, Venturi meter, and current meter. They also include the salt-velocity method and the Gibson pressure method, which have been widely used for efficiency tests. The list would not be complete without adding the stream-gauging method, which is in general use on rivers. Some of these methods have definite applications, together with definite limitations, as devices for obtaining a continuous record of discharge through a hydraulic turbine or power plant. Obviously, the salt-velocity and Gibson pressure methods, and the use of the current meter, are too complicated for continuous service and must be limited to testing purposes.

The weir is an ideal device for obtaining continuous records of discharge, but it is limited to small quantities, and, even then, it requires a loss of head, which is uneconomical. The Venturi meter may also serve for continuous records where its installation is feasible and economical, but it will be limited generally to relatively small quantities. The stream-gauging method is suitable where a satisfactory control is available and will give good results if used intelligently. Such a station, however, can only measure the total flow and cannot segregate the discharge to various turbines, or to flow over the spillway.

NOTE.—The paper by Ireal A. Winter, Assoc. M. Am. Soc., C. E., was published in April, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[6] Engr. (Young & Stanley, Inc.), Muscatine, Iowa.

[6a] Received by the Secretary April 29, 1933.

Unfortunately, none of the aforementioned devices is fitted for general application to individual turbines for the purpose of obtaining continuous records of discharge. For such purposes, a device must give a single, unvarying indication for any given discharge and must be susceptible of calibration in terms of true discharge. Such requirements lead into the field of the so-called "index" methods, in which the pressure differential between two points, or the velocity at some one point, is related definitely to the total discharge. Within recent years, considerable attention has been given to the development of such "index" methods for use in obtaining continuous records and as a check against more elaborate test methods.

The use of pressure differentials appears to offer greater possibilities than the use of velocities because pressures are more stable than velocities, and the instrument for measuring pressures—the piezometer—is a more simple and more reliable device than the instrument for measuring velocity—the Pitot tube, or the current meter.

The Winter-Kennedy method is an ingenious application of the pressure differential method to scroll-case installations. This paper outlines the design and application of such equipment in an interesting manner. The writer believes that the principles of this method may be extended to a wider application to fit cases other than those having closed scroll-cases. Whenever there is a flowing water, there will be pressure differentials between various points which will result from friction losses, changes in velocity, or centrifugal action. In all these cases, the pressure differential will probably be a function of the square of the discharge. It is probable that differentials resulting from changes in velocity, or centrifugal action, will be more reliable and consistent than those resulting from friction losses, as the friction losses tend to vary with time

Wherever pressure differentials are of sufficient magnitude to allow satisfactory measurement, and wherever they are a definite function of the discharge, they may be utilized as an index of the discharge; with proper calibration, they may be used to express the total discharge. No doubt, in many instances, these differentials will have to be measured in inches of water rather than in inches of mercury, and equipment will have to be arranged to fit the particular application.

As an example, assume an intake structure to a tunnel with a water velocity of 2 ft per sec immediately behind the trash tracks, and a velocity in the tunnel of 8 ft per sec. This change in velocity involves a pressure differential of nearly 1 ft of water which will occur in a relatively short distance. Such a differential is capable of measurement. Other applications might be feasible at bends of penstocks or pipe lines, and at various points on the turbine speed-ring, guide vanes, or gates of units having an open flume setting.

Of course, such installations will require careful investigation and experimentation to obtain reliable results. It is to be hoped that further study along these lines will develop a wider application for the principles which the author has applied so successfully to hydraulic turbines with scroll-cases.

R. C. JOHNSON,[7] ASSOC. M. AM. SOC. C. E. (by letter).[7a]—The author has presented clearly the mathematical reasoning and experimental evidence leading to very definite conclusions. An understanding by the profession of the operation and installation of this improved type of flow meter should certainly lead to its more general adoption.

In addition to the operating advantages mentioned, such flow meters should offer particular advantages when used in connection with adjustable-blade propeller turbines, where the operating conditions sometimes vary through much wider limits than for ordinary installations. Continuous flow records for the purpose of checking efficiencies would be especially useful in connection with hand-operated adjustable-blade runners. The use of an efficiency curve as the sole guide in the operation of a unit is indeed inadequate. The character of an efficiency curve is by no means constant even under the same apparent conditions of operation. An easy method of checking the efficiency quickly, under any combination of operating conditions, would often lead to the location of sources of waste which would otherwise go unnoticed.

The author points out the eccentric loading condition due to differences in pressures over the scroll-section and the resulting bending moments produced by it. Expressing the pressure difference by Equation (3), with C having a range of values from about 0.75 to 1.25, it can be seen that the pressure difference between any two points in the scroll-case could scarcely be sufficient to cause undue annoyance in the structural design, yet stresses produced by such pressure should certainly be taken into consideration. Fig. 8 shows the differential pressure between Piezometers Nos. 1 and 4 (Fig. 5) to be about 22 in. of mercury (approximately 25 ft of water), for maximum discharge conditions. While moments produced by such eccentric loads could not be enormous, a knowledge of their magnitude and distribution throughout the entire scroll-section, based on experimental investigations, would remove considerable guesswork now necessarily used by the structural designer, resulting in a more economical use of materials.

E. A. DOW,[8] M. AM. SOC. C. E. (by letter).[8a]—An interesting presentation of a most useful method of metering the discharge of large hydraulic turbines is presented in this paper. Undoubtedly, it will be applied with increasing frequency in future hydro-electric practice.

The turbines of the Comerford Plant of the New England Power Association System (15-Mile Falls development, near Littleton, N. H.), are similar to that from which the data shown in Fig. 8 were obtained, and the results of the calibration test at this plant may be of interest for comparative purposes.

The Comerford turbines are equipped with a number of piezometers to provide a wide variety of differentials for use as mutual checking systems in connection with the measurement of turbine discharge by the salt-velocity

[7] Assoc. Prof., Civ. Eng., Univ. of South Carolina, Columbia, S. C.

[7a] Received by the Secretary June 8, 1933.

[8] Designing Engr., New England Power, Eng. & Service Corp., Boston, Mass.

[8a] Received by the Secretary July 20, 1933.

method. Among these were four piezometers installed in accordance with designs furnished by Mr. Winter and corresponding to Piezometers Nos. 1, 2, 3, and 4, shown in Fig. 5. In Fig. 11, the performance of these piezometers is presented in logarithmic form similar to Fig. 8, except that the differentials are expressed in feet of water. The data are for the piezometers corresponding to Piezometers Nos. 2 and 4 of Figs. 5 and 8, these being the ones observed during the salt-velocity test.

The solid line ($Q = 1\ 355H^{0.490}$), represents the calibration formula derived from twenty test runs. This was arrived at by least squares to eliminate

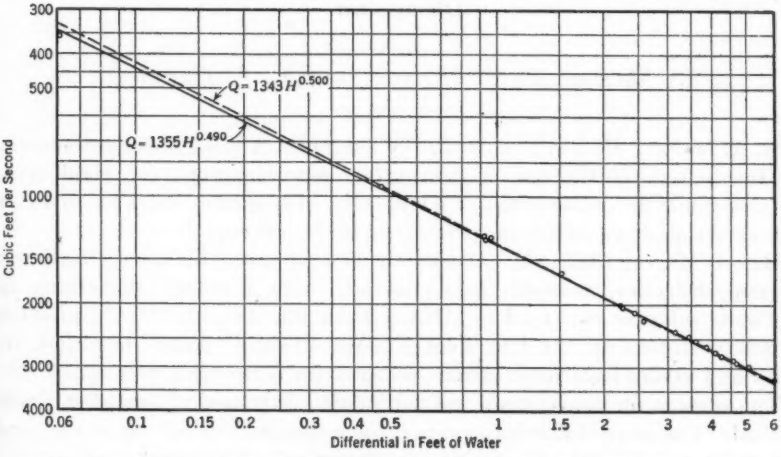

FIG. 11.—PERFORMANCE OF PIEZOMETERS IN TURBINES OF COMERFORD PLANT.

personal bias, and assumes equal probability of errors in determining the differentials and the discharges. The dotted line ($Q = 1\ 343H^{0.500}$), represents a square law calibration, with the coefficient (0.500), assumed to make the average of the ratios of computed discharge to salt-velocity discharge equal to unity.

It will be noted that the two calibrations intersect at about 2 100 cu ft per sec, or 65% of capacity; and that the square law discharge is 101% of the logarithmic discharge at 100% of capacity, and 96% at 10% of capacity. It is obvious that the square law coefficient may be slightly modified to cause the line to pass through some preferred point, sacrificing the average agreement with the salt velocity to some extent. The calibration thus reduces to a choice of assumptions, either that the square law is inviolate, or that, on the average, the measured discharges are correct.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WIND STRESSES BY SLOPE DEFLECTION AND CONVERGING APPROXIMATIONS

Discussion

BY MESSRS. G. A. MANEY, AND L. E. GRINTER

G. A. MANEY,[3] M. AM. SOC. C. E. (by letter).[3a]—A brief statement concerning the history of the use of converging approximations for rigid frame analyses might be interesting. Undoubtedly, this system dates about as far back as the problem of solving simultaneous equations.

Before March, 1915, when the writer first proposed his generalization of the slope-deflection method,[4] Mohr's method, with a rapidly converging approximate solution, was used by Allston Dana, M. Am. Soc. C. E., to obtain secondary stresses in the long-span Kenova Bridge.[5] Since that time, the writer and others have used various schemes for converging solutions.

The reasons for convergence are not subtle mathematical ones, but simply physical. The slope-deflection equation for a member loaded transversely and with end rotations, but no rotation of member as a whole, is:

$$M_{AB} = M_{F/AB} - K\,(2\theta_A + \theta_B) \qquad (14)$$

in which, the coefficient, K, varies as $\frac{I}{L}$ and θ is multiple of angle.

This equation written in long hand states that the final bending moment, M_{AB}, at the end, A, of any member, AB, is equal first to the value, $M_{F/AB}$, which is the moment occurring at the end, A, due to intermediate transverse loads when neither Joint A nor Joint B are allowed to rotate. Then, as Joint A is allowed to rotate through the angle, θ_A, a moment equal to $(K)\ (2\ \theta_A)$ is superimposed on top of $M_{F/AB}$ at the end, A. Incidentally, this angle, θ_A, induces only one-half as much moment at the end, B. Hence, when the end, B, is rotated through the angle, θ_B, only $(K)\ (\theta_B)$ is induced at the end, A.

The minus sign is really not important, but as stated in Equation (14), means that if θ_A is a clockwise rotation of the joint, A, the induced moment tends to rotate the joint, A, counter-clockwise.

NOTE.—The paper by John E. Goldberg, Jun. Am. Soc. C. E., was published in May, 1933, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

[3] Prof., Structural Eng., Northwestern Univ., Evanston, Ill.

[3a] Received by the Secretary June 2, 1933.

[4] *Studies in Engineering No. 1*, Univ. of Minnesota, 1915.

[5] "Statically Indeterminate Stresses," by J. I. Parcel and G. A. Maney, Members, Am. Soc. C. E., N. Y., 1926, p. 330.

The real and only reasons for convergence are contained in the expression,

$$M_{AB} = -(K)(2\theta_A + \theta_B) \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which, M_{AB} is used as a correction or addition to an original or known starting value which is a function of the member rotation as a whole (with the zero member end rotations) for wind load and secondary stress problems, and which is M_F (the well known "fixed beam" moment) for ordinary building frames under gravity loads.

The reasons for convergence may be stated, as follows:

(1) In one extreme case, where the end, B, is free to rotate, an angular change, θ_A, applied at the end, A, results in only one-half as large an angular change, θ_B, at the end, B.

(2) In the other extreme case, for the end, B, where it is fully fixed, an applied bending moment, M_{AB}, at the end, A, results in only one-half as much bending moment, M_{BA}, at the end, B.

(3) When corrections in either angular change or bending moment are applied to either the original zero angular values or the original M_F-value, such correction (or error in such correction) is felt by only one-half the amount of the correction at joints once removed (opposite end); by only one-quarter at joints twice removed; and by only one-eighth at joints three times removed (three member lengths away).

Examining Equations (14) and (15) it will be seen that whether one goes from the starting point, $M_{F/AB}$, to the final value, M_{AB}, with a series of well controlled "guesses" by means of correcting M_F directly or by means of correcting θ-values, no essential difference in method is involved. Advocates of the so-called "moment-distribution" method prefer correcting M directly and it is possible that this procedure will alleviate the strain on the imagination which might be necessitated to visualize the physical distortions of the structure.

The writer wishes to commend Mr. Goldberg on having produced what is, in his estimation, the most expeditious convergence scheme yet applied to a frame under wind loads.

The writer and Mr. Goldberg have made a comparison between the moment-distribution "method" and the angular correction method used in the present paper.[6] It will be clearly seen by this comparison that they merely typify two methods which can be used advantageously to travel from the "fixed beam" moment of the slope-deflection method to the final moment by a quick and self-checking series of converging approximations.

L. E. GRINTER,[7] ASSOC. M. AM. SOC. C. E. (by letter).[7a]—The author has presented a method of solving for the joint rotations and lateral deflections of a building frame acting under wind load by successive substitution in approximate and exact equations. The title of the paper will lead most

[6] "Simplified Methods for Analyses of Multiple-Joint Rigid Frames", by G. A. Maney, M. Am. Soc. C. E., and John E. Goldberg, Jun. Am. Soc. C. E., *Bulletin*, Northwestern Univ., School of Eng., 1932.

[7] Prof. of Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[7a] Received by the Secretary June 5, 1933.

readers to expect a closer relationship to the general slope-deflection method than really exists. The author does use the general slope-deflection equations in the derivation of his working formulas (Equations (10) and (11*c*)), but, as the writer has shown,[8] such working formulas are more readily obtained by area moments. However, since the unknowns to be determined are rotations and deflections rather than moments, the name, "slope deflection," is properly descriptive of the method.

Thirteen steps are listed in the paper before one arrives at a final set of joint rotations and lateral deflections. Actually, this outline represents more than thirteen operations since Step (11) involves the repetition of three previous steps and Step (13) involves the repetition of two. The author indicates that Step (13) is usually unnecessary and that Steps (11) and (12) may be omitted for regular frames. Obviously, a criterion is found in Equation (11*c*). If this equation accurately predicts the lateral deflections of the frame (which will be true for frames of reasonable regularity) then the first approximations of the θ-values and R-values will be sufficiently exact, and the analysis will be completed by the first ten steps. Note, however, that in any case one must then compute the moments from the rotations and deflections by substitution into the general slope-deflection equations.

In order to apply the entire fourteen steps of the procedure to the lower four stories of an irregular building having an outline similar to Fig. 2, the writer estimates that one would have to compute approximately two hundred preliminary and final values of rotation, deflection, and moment by substitution into equations varying in complexity from Equation (10*a*) to Equation (12). Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings, has estimated the time required by its proposed method to be 10 min per member,[9] or approximately 5 hr for this structure. This time limit has been reduced by the writer by the use of a simplification of the method proposed by the Sub-Committee. It would seem unlikely that the author's method could be applied within this time limit. The importance of time limit in wind stress calculations has been much emphasized recently in *Proceedings*.[10]

The author's method in common with most methods based directly on simultaneous equations must be classed as mathematical rather than visual. For instance, the various steps of the process cannot be visualized as directly associated with the physical action of the structure. Hence, the operator must depend entirely either on memory or on printed formulas when applying the method. In order to check this statement the writer assigned the analysis of a frame by this method to one of his graduate students. The student followed the outline step by step and obtained a satisfactory analysis. However, finding nothing physical about the process to tie the various steps together, he neither understood nor retained a knowledge of the procedure.

The writer feels that the most valuable part of the author's paper is the discussion of the composite bent. Since the relative amount of load resisted

[8] *Proceedings*, Am. Soc. C. E., January, 1933, pp. 20 and 22.
[9] *Loc. cit.*, February, 1932, p. 218.
[10] *Loc. cit.*, April, 1933, p. 699.

by each bent is measured by its resistance to deflection, it is natural that a slope-deflection method should simplify the problem of the division of load between bents. The writer is inclined to believe that this problem of load distribution has received less than its proper amount of attention from practical designers. There seems to be an unjustifiable belief that distribution of load is of little importance if the total resistance of all bents is adequate. Observe the careless use of diagonal bracing and knuckle connections in adjacent bents. In this same connection, the author mentions torsion in the asymmetrical frame, but offers no solution to the problem. This is one of the most serious problems facing the structural engineer and one which has received but little attention.

The author finally discusses the problem of secondary wind stresses or, more exactly, the effect of column deformations upon wind moments. Clearly, his slope deflection method is very poorly adapted to the solution of this problem. What excuse could there be for one to use joint movements to calculate joint rotations and lateral deflections when these must then be re-used to calculate moments? The direct procedure is to use joint movements to calculate fixed-end girder moments which are then easily changed into correction moments by the general idea of moment distribution. The original advantage of the slope-deflection method, namely, that there are fewer θ-values and R-values to handle by simultaneous equations than there are joint moments (although the number of equations is the same), is no longer an important advantage when the analysis is performed by successive corrections.

The author should be complimented for his attempt to reduce the labor of wind-stress analysis by the slope-deflection method. Despite the fact that he believes that it will consume more time than other methods of successive corrections, the writer realizes that there are many engineers who will find it useful. Particularly, those older engineers who have made wide use of the general slope-deflection method will appreciate the availability of this simpler tool. The paper is also of service in calling the attention of the profession again to the importance of wind-stress calculations in controlling the design of tall buildings. Only by continued emphasis can the less informed members of the profession be made to realize that there is no place to-day for the crude approximations that were the only reasonable tools available a few years ago. The writer was particularly pleased to note the author's remark that his method does not place "too much confidence" in the belief that "points of contraflexure tend to approach the mid-lengths of the members." There seems to be an over-confidence among many writers that the ideal of economic design is to accomplish this result.

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DISCUSSIONS

EARTHS AND FOUNDATIONS

PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY MESSRS. L. C. WILCOXEN, H. DE B. PARSONS, WILLIAM P. KIMBALL, AND T. A. MIDDLEBROOKS

L. C. WILCOXEN,[6] Assoc. M. AM. SOC. C. E. (by letter).[6a]—The progress report of the Special Committee on Earths and Foundations is a remarkable contribution to theoretical soil mechanics. It is to be hoped that the truths

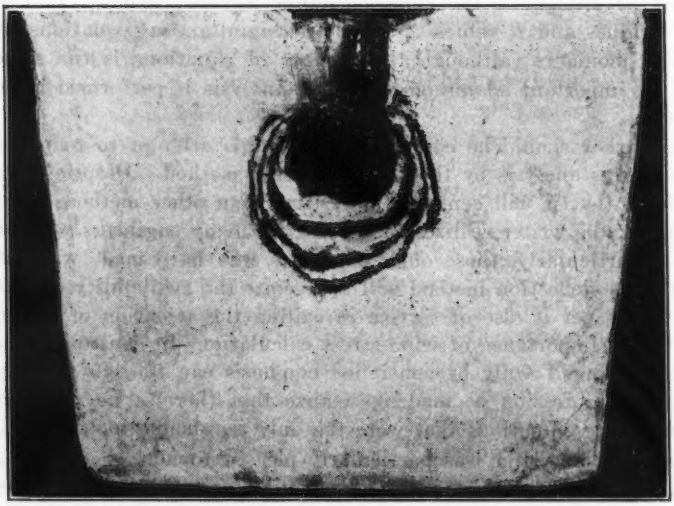

FIG. 42.—EXTRUDED BULB OF DISPLACEMENT IN A THIN LIME PASTE.

therein will rapidly supplant the false rule-of-thumb methods of foundation design, so frequently practiced to-day.

The writer observes, however, that certain necessary qualifications for the use of the Boussinesq method of analysis are inferred, but not rigidly adhered

NOTE.—This Progress Report of the Special Committee on Earths and Foundations was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in May, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[6] Asst. Civ. Engr., City Engr.'s Office, Detroit, Mich.

[6a] Received by the Secretary June 21, 1933.

to. To use the Boussinesq method, the loading must be flexible. Rigid footings give markedly different stress distributions while semi-rigid footings necessarily yield hybrid results.

Fig. 42 shows one of the bulbs of displacements made by the writer in a thin lime paste. Similar bulbs were made in a stiff undisturbed blue clay and in a loosely packed damp mortar sand. In each case the bulbs were formed by extruding small batches of the same material through the bottom of a rigid tube 1 in. in diameter. The extrusion was effected by means of a plunger. Alternate batches of the material were mixed with lampblack

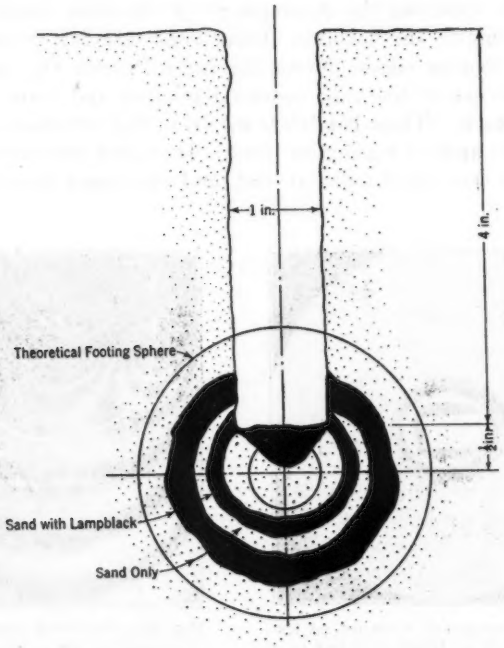

FIG. 43.—TRACING OF AN EXTRUDED BULB OF DISPLACEMENT IN A LOOSELY PACKED DAMP MORTAR SAND.

in order to show the distribution after extrusion. The resulting bulbs were parted on the vertical center line and photographed. The photograph of the sand bulb proved indistinct, so it was traced as shown in Fig. 43.

These simple tests show (as the Boussinesq theory states and as building settlements confirm) that with a flexible load the differential settlements are progressively greater from the load perimeter toward its center.

Fig. 44 shows a horizontal section of a bulb in a lime paste. The section was taken at the bottom of the rectangular tube through which it was extruded. The form of this bulb harmonizes with the Boussinesq-derived stress distribution in Fig. 16 of the report.

Fig. 35 of the report shows a comparatively rigid steel footing on bakelite. The radically different stress distribution of the rigid footing warrants elucidation. The line of maximum stress, with this type of footing, extends from the edges, through an arch which crosses the center line at the depth approximately 0.7 of the footing width.

Fig. 45 is from a photograph by Dr. Max Frocht,[7] of the Carnegie Institute of Technology, on which the writer has indicated the inverted arch. With a three-dimensional footing, it would be an inverted dome.

The important fact to note is that the soil stresses diminish, not only outside this arch, but also within it. This may be confirmed, as was done by the writer, by watching the development of the color bands, as such a load is slowly applied to the bakelite. Color rings are first generated simultaneously at the footing edges, developing inward along the arch of maximum stress. At the center line they contact the upper and lower portions, parting to form two bands. These bands are shown in Fig. 45 where they are parallel, roughly, to the arch of maximum stress. Increased pressure causes the lower band to move downward and outward, and the upper band to move upward and inward.

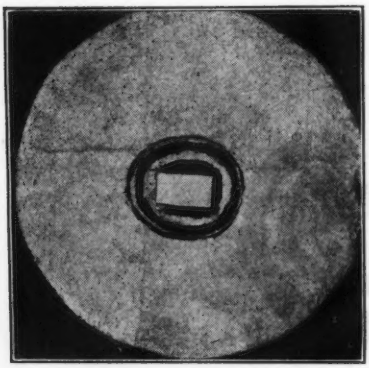

FIG. 44.—HORIZONTAL SECTION OF EXTRUDED BULB OF DISPLACEMENT IN A THIN LIME PASTE.

FIG. 45.—PHOTO-ELASTIC STRESS DISTRIBUTION OF A RIGID STEEL FOOTING ON BAKELITE.

Such a test demonstrates that with a rigid footing the soil pressure, on the footing itself, is greatest at the perimeter, decreasing toward the center. This, of course, may be deduced from the Boussinesq theory. It is the necessary complement of the fact that a uniform (flexible) load causes increasing settlements toward the center. Below the inverted arch of maximum stress, the Boussinesq theory possibly applies to rigid as well as flexible footings. This explanation is, therefore, not in conflict with, but rather supplements, the report.

In the paragraph referring to Fig. 36, it is stated: "The upper plate represents a rigid footing—the lower plate, the elastic underground or sup-

[7] "Kenemotography in Photoelasticity," by Dr. Max Frocht, *Transactions*, A. S. M. E., Vol. 54, A P M 54–9, p. 89.

porting medium." Obviously, both are elastic, the bakelite footing, due to its own deformation, transferring what approaches a uniform load. The result is a stress distribution of the Boussinesq pattern.

H. DE B. PARSONS,[8] M. AM. SOC. C. E. (by letter).[8a]—This is a very interesting progress report and, in the writer's opinion, one of the most useful that has been published by a committee of the Society for some time. It contains considerable information for designs of foundations on compressible soils.

Some years ago, the writer was retained by a contractor to express an opinion on foundation conditions for hospital buildings which were under construction by a State. The writer had borings made which showed that the rock was about 25 ft below the foundation footings. Above the rock there was a layer of soft shale; then a layer of fine sand; then a layer of coarse sand; then a layer of fine sand and red clay, with pockets of yellow and slate-colored clay; and a covering of top-soil and forest humus. The site was swampy, although there had been little rain prior to the examination. The buildings were designed for a loading of about 2 tons per sq ft, on footings of concrete in shallow trenches. The trenches had tile pipe drains laid in broken stone, but the invert of the drains was above the bases of the footings. Had the information in this report been available and had the engineers of the State used the information intelligently, the footings of the buildings could have been designed to better advantage.

The bearing quality of the soil was tested by constructing a loading apparatus, having a bearing surface of 1 sq ft, which supported a loading platform on which was placed 4 000 lb of dead weight. Where the ground was drained by sub-surface ditches and tile pipes below the test bases, there was a settlement, in Test No. 1, of 0.8 in. in 9 days, and, in Test No. 4, of 0.73 in. in 6 days. Where the ground was not drained, there was a settlement, in Test No. 2, of 10.9 in. in 12 days and, in Test No. 3, of 5.64 in. in 4 days.

Engineers have assumed for many years that the pressure on soils extended outward from the edge of the foundation at 30° from the vertical, or 60° from the horizontal, without reference to the character of the soil. This old rule of thumb is still in use. Recently, the bulb theory of pressures has been developed, and research has shown that a load is not uniformly distributed over the under side of a foundation footing, but is heaviest under the point of application of the load (which is generally at the center), and diminishes toward the edges.

While the report states that the pressure distributions are independent of the type of material and involve no elastic constants, the writer offers a physical explanation for these bulb contours of uniform pressure. Each bulb of uniform pressure intensity represents a lower pressure according to its position outward, or away from the center or place of maximum load. The tangent to an outside contour, starting at the edge of the foundation, probably makes an angle with the vertical equal to the angle of rupture of the

[8] Prof. Emeritus, Rensselaer Polytechnic Inst., Cons. Engr., New York, N. Y.
[8a] Received by the Secretary June 22, 1933.

soil. It is the same angle as would be found between a vertical wall confining the soil and the plane of rupture. The angle of rupture depends on the internal friction of the material. This idea is illustrated in Fig. 46, in which *B-A-C* is the angle of the plane of rupture. At increasing distances from *A*, the unit pressure along the line, *A-C*, diminishes; also, at increasing depths below the surface, the angle of internal friction increases in a soil owing to the weight of the soil material above.

Consequently, due to the diminishing pressure and the decreasing angle of rupture, a uniform pressure contour would curve downward, away from the line, *A-C*. Continuing this reasoning, the angle of rupture would become zero, and the pressure would equal the shear of the soil and the curve would

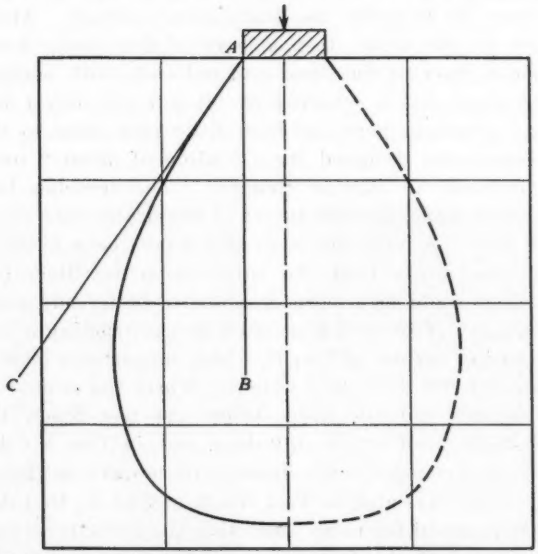

FIG. 46.—EXPLANATION OF BULB FORMATION.

become vertical. Again, continuing, the soil shear (now strengthened by the greater depth) would exceed the pressure, represented by the curve, with the result that the pressure would simply compress the soil. The bulb curve then would flatten toward the horizontal and meet a corresponding curve from the opposite side of the foundation, thereby producing the completed bulb.

When the soil contains moisture, the loading will force some of the water outward, that is, from under the foundation, and, in consequence, may change the shape of the uniform pressure contour sketched in Fig. 46, because the water will change the position of the surface of rupture and the shearing strength of the soil

The contour sketched in Fig. 46 is such that the pressure along it is sufficient to cause rupture at the angle, *B-A-C*. Any contour outside this one would represent a uniform pressure too low to cause rupture. The material in such

an outside zone would be affected by the particles trying to hold themselves together, but nothing more. Any contour inside the one referred to simply represents compression of the material.

The writer would like to know if the research work of the Committee determined this angle, *B-A-C*; and, if so, how it compared with the angle of rupture as defined and used in applying soil pressures behind a bulkhead?

WILLIAM P. KIMBALL[9], JUN. AM. SOC. C. E. (by letter)[9a].—In this progress report the Committee has made available a wealth of general and specific information for the guidance of engineers interested in predicting the settlement of structures founded on deep beds of compressible soils. It has been the writer's privilege during the two years—1931 and 1932—to be engaged in subsoil analyses for both the San Francisco-Oakland Bay Bridge and the Huey P. Long Bridge, across the Mississippi River, at New Orleans, La., where the piers are designed to rest on or above deep deposits of compressible materials. In the case of the San Francisco-Oakland Bay Bridge the soil analysis was conducted prior to the location and design of the piers and was useful in determining safe landing elevations, pier sizes, and general design features. The soil analysis for the New Orleans Bridge was conducted subsequent to the design of the piers as a check against a few tests performed in 1926 by Charles Terzaghi, M. Am. Soc. C. E., prior to the selection of landing elevations and the design of the piers. In predicting the settlements of these structures the writer made wide use of the material presented in Professor Gilboy's report to the Committee on the settlement of an existing structure and of other information published for the first time in this report. Other theories and devices which have been helpful in predicting settlements may be of interest to foundation engineers and are presented herein.

The determination of stress distribution under foundations even by exact theories such as Boussinesq's and the method of plane sections is generally acknowledged to give only approximations of the stresses actually existing because of the uncertainties of stress intensities at the loaded surface and the non-isotropic properties of the underlying soils. As illustrated by Professor Terzaghi's water tank, described in "Case F" of the report, a uniform load transmitted through a very yielding distributing mat results in differential settlements producing the "dishing" effect commonly observed in extensive, flexible mat foundations. Conversely, it has been shown by Mr. Arthur Casagrande[10] and other investigators[11] that the stress distribution under a rigid plate assumes an approximately parabolic form. In computing the stresses under bridge piers where heavy seal mats of tremie concrete provide a substantially rigid bearing surface this irregular form of distribution should be assumed. The work involved in determining

[9] Instr. of Civ. Eng., Thayer School of Civ. Eng., Dartmouth Coll., Hanover, N. H.; formerly, Soils Engr., with Moran & Proctor, New York, N. Y.

[9a] Received by the Secretary June 27, 1933.

[10] "The Structure of Clay and its Importance as a Foundation Material", *Journal*, Boston Soc. of Civ. Engrs., Vol. XIX, No. 4, April, 1932, p. 168.

[11] Most recently, M. A. Sadowsky, in Paper No. A P M–54–21, of the Am. Soc. Mech. Engrs.

stresses by the Boussinesq theory, using the summation of effects of finite areas, is extremely arduous. The writer has adopted a method suggested by Mr. G. W. Glick, of the firm of Moran and Proctor, of substituting for the actual loaded area an equivalent circular area under the center of which the effects may be readily integrated and expressed by the formula:

$$p_z = p\left[1 - \frac{1}{\left[1 + \left(\frac{D}{2z}\right)^2\right]^{\frac{3}{2}}} \text{ exp.}\right] = p\,K' \quad \ldots\ldots\ldots \quad (37)$$

in which, in addition to the notation of the report: p_z = the vertical component of stress; p, the uniform load applied to the bearing area; D, the

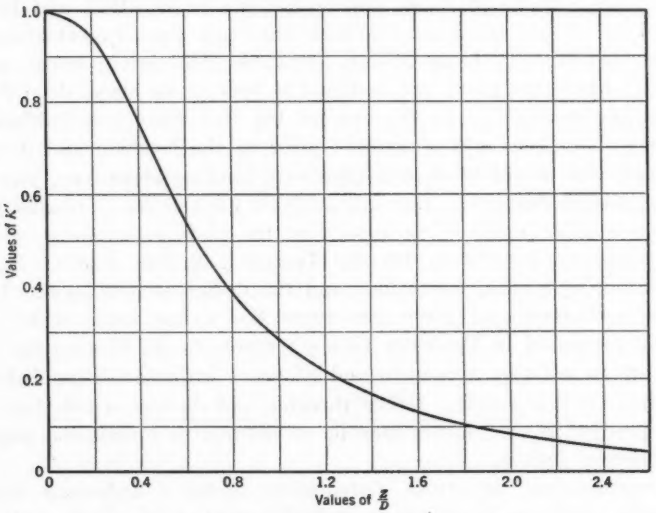

FIG. 47.—VALUES OF K' IN EQUATION (37).

diameter of the bearing area; and z, the depth of the point at which the stress is to be computed below the bearing surface.

Fig. 47 shows the relation between K' and $\frac{z}{D}$. A comparison of the results obtained by a summation of the effects of finite areas of Pier V of the New Orleans Bridge, a rectangular bearing area approximately 60 by 90 ft, and by integrating under the equivalent circular area, is shown in Fig. 48. It will be observed that the maximum variation occurs 50 ft below the loaded surface and amounts to less than 3 per cent. Since the settlement of a rigid foundation will be caused by consolidation under the average stress under the entire area, rather than under the maximum stress indicated under the center, the writer has calculated these average intensities which are compared with the center line stress in Fig. 49. Although this method of determining effective stress distribution is only approximate the accuracy of the calculated stresses is probably sufficient for the purposes of a settlement analysis.

The Committee, under "The Mechanics of Settlement in Clay," states that "clay differs from silt and sand as a foundation material in that settlement takes place through the consolidation of the clay mass as the water is squeezed out under an applied load." The writer feels that settle-

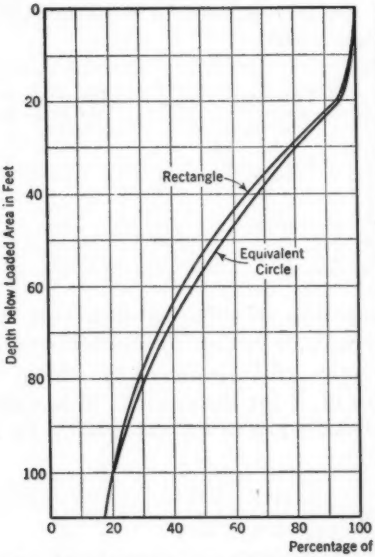

FIG. 48.—STRESS DISTRIBUTION UNDER UNIFORM LOAD.

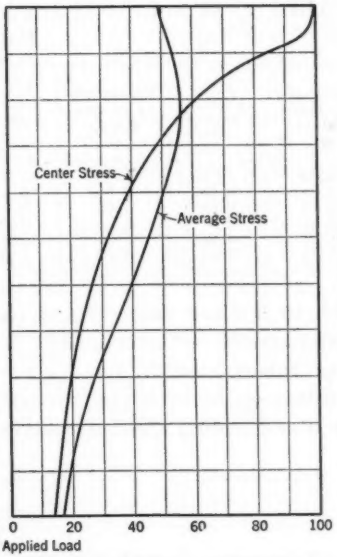

FIG. 49.—STRESS DISTRIBUTION UNDER RIGID PLATE.

ment may also be caused by the "consolidation" of deep deposits of saturated silt, having a honeycomb structure, as the water is squeezed out. Is there any evidence that this occurrence is impossible or improbable?

In the use of the consolidation equations the writer has derived the equation for J, Equation (31) in the report, with the following result:

$$J = \frac{\log \dfrac{\pi\,(1+v)}{2\,(\pi-2)\,v+4}}{\log \dfrac{\pi}{4}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

It will be observed that Equation (38) differs from that given by the Committee only in that the numerator is inverted. The resulting influence table for J and v is identical with that published by the Committee, except that values of J are positive, whereas the values calculated by Equation (31) are negative.

In Table 5 the Committee has tabulated values of Q and N for cases of uniform pressure (Case 0), and triangular pressure (Case 1), where the pressure increases from zero at the drainage surface. The writer has had occasion to apply the consolidation equations to a third common case in which the pressure decreases with the distance from the drainage

surface, where v is greater than one. As an addition to Table 5 the writer offers values of Q and N, in Table 7, for Case 2, in which the pressure decreases from a finite value at the drainage surface to zero at the limiting impervious surface where v equals ∞.

TABLE 7.—VALUES OF Q AND N FOR USE IN SOLVING EQUATION (28) AND EQUATION (30)

Percentage of consolidation Q	Case 2, value of N	Percentage of consolidation, Q	Case 2, value of N	Percentage of consolidation, Q	Case 2, value of N	Percentage of consolidation, Q	Case 2, value of N.
0.00	0.000	0.55	0.32	0.30	0.06	0.85	1.36
0.05	0.002	0.60	0.42	0.35	0.07	0.90	1.77
0.10	0.005	0.65	0.54	0.40	0.13	0.95	2.54
0.15	0.01	0.70	0.69	0.45	0.18	1.00	∞
0.20	0.02	0.75	0.88	0.50	0.24	....	....
0.25	0.03	0.80	1.08	....	....	....	....

If, in a given case, with a trapezoidal distribution such that the ratio of the pressure at $z = 0$ to that at $z = h_o$, is v, then the interpolated value of N, in Tables 5 and 7, for a given value of Q, is found by taking J' times the difference between the two values of N for the given Q from Case 2 and Case 0 and adding this interpolated difference to the value of N for Case 2. Let,

$$J' = 1 - \frac{\log \dfrac{\pi\,(1 + v)}{2\,(\pi - 2)\,v + 4}}{\log \dfrac{2\,(\pi - 2)}{\pi}} \qquad \text{(39)}$$

The values of J' for corresponding values of v are as follows:

Value of v	Value of J'	Value of v	Value of J'
1.0	1.0	6.0	0.34
1.5	0.83	7.0	0.30
2.0	0.71	8.0	0.27
2.5	0.62	9.0	0.25
3.0	0.55	10.0	0.23
3.5	0.50	12.0	0.20
4.0	0.45	15.0	0.17
4.5	0.42	20.0	0.13
5.0	0.39	....	....

The settlement analysis of Pier E-3 of the San Francisco-Oakland Bay Bridge illustrates the application of the factor, J', lying between Cases 0 and 2 to the consolidation equations. This pier as designed will require the deepest open-dredged caisson ever built, landing 220 ft below mean low water. Bed-rock was located by borings at about —296 ft. Overlying the rock is a blanket of clay about 33 ft thick, in contact at the top with a stratum of coarse sand and gravel, providing a drainage course for the clay

deposit below. The stress imposed by the pier at the top face of the clay is about 1.5 tons per sq ft and at the bottom of the clay the stress increment is 1 ton per sq ft. The ratio, v, therefore, is 1.5, corresponding to a value of J' of 0.83.

In discussing the application of these factors to the consolidation equations it is interesting to note that when a stratum of clay is subjected to a substantially straight-line stress increase from top to bottom and is in contact with drainage courses at both faces, the effects of applying the factor, J, to the upper half of the stratum and J' to the lower half practically balance, and the consolidation of this stratum proceeds at substantially the same rate as if it were subjected to a uniform stress increment throughout its depth.

The writer with the counsel of Daniel E. Moran and G. L. Freeman, Members, Am. Soc. C. E., has developed and operated a consolidation device which is believed to have certain improvements over devices previously used for these tests. Sketches of the device and rings used for introducing the undisturbed samples are shown in Fig. 50. During a test the soil sample

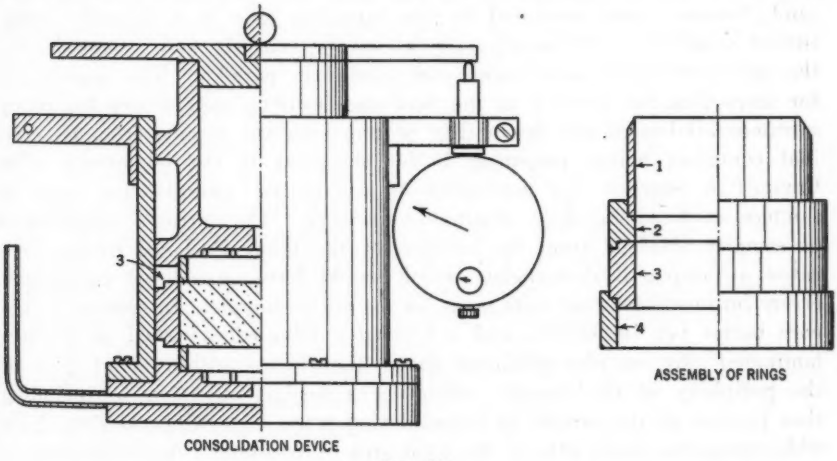

FIG. 50.

is confined in the removable testing ring (3) having an inside diameter, as originally designed for testing samples taken from a 3-in sampler operating through a 4-in. casing of $2\frac{1}{2}$ in. and a depth of $\frac{1}{2}$ in., $\frac{3}{4}$ in., or 1 in., as desired. In order to reduce the error due to side friction between soil and ring the testing ring floats unsupported except by adhesion to the clay sample. In testing undisturbed samples, the removable soil chamber insures plane, parallel upper and lower faces, which are struck off with a straightedge, and affords a direct check of the compression indicated by the deflection dial readings by careful weight observations made before and after the test. The "Assembly of Rings" shown in Fig. 50 illustrates the method of entering the soil specimen into the testing ring (3). A specimen of soil is first cut from the undisturbed boring sample by means of the cutting

ring (1), from which it is pushed by a plunger through the transfer ring (2) into the testing ring (3). The writer has found it convenient to conduct these operations on a hand-operated bench drill to insure parallel motion and to prevent rocking and resultant irregularity in the specimen. The sample is then struck off at both faces and is ready for testing. This procedure is simple and quick, and gives more satisfactory and reliable results than any other method the writer has used for testing undisturbed clay samples.

The writer concurs with the Committee in emphasizing the importance of carefully taken undisturbed soil samples where soil analyses are to be conducted. In directing a series of borings for undisturbed samples at the location of the bridge in New Orleans recently the writer supervised the building of sampling tubes similar in principle to that shown by the Committee in Fig. 25. Using these tubes, samples, 3 in. in diameter, were readily obtained through a 4-in casing, from depths as great as 365 ft below river level. The chief purpose of this boring program was to obtain undisturbed samples of clay for laboratory analysis. Many samples of very fine clean sand, however, were recovered in this sampling tube in a virtually undisturbed condition. All samples for laboratory analysis were removed from the split core barrel and immediately sealed in paraffin. The opportunity for inspecting the samples in the field and insuring satisfactory laboratory specimens indicated the desirability of removing the sample from its original container before preparing it for shipment to the laboratory. The Committee suggests the possibility of conducting consolidation tests on samples confined in their original containers. The writer's observations of samples obtained from the location of the Huey P. Long Bridge indicated a peripheral disturbance which would have introduced appreciable errors into consolidation tests made on the entire sample. In spite of a thin core barrel ($\frac{5}{16}$ in. thick), and a beveled cutting-edge turned in slightly, laminated clay samples exhibited sharp disturbance within about $\frac{1}{8}$ in of the periphery of the sample, indicating a serious objection to including that portion of the sample in consolidation tests. A peripheral ring, $\frac{1}{8}$ in. wide, comprises about 16% of the total area of a sample 3 in. in diameter.

The Committee refers to the necessity of taking undisturbed samples from the bottom of a dry hole. The writer's experience has shown that it is frequently impracticable to attempt to keep a deep boring dry. Theoretically, also, such a procedure is unwarranted. In conducting the borings previously mentioned the casing was washed down to the elevation of the sample to be taken. The bottom of the hole was then flushed clean under low pressure and the sampling tube driven ahead of the casing. Inasmuch as the top and bottom 6 in of each sample are discarded, and only the middle portion prepared for shipment to the laboratory, the writer feels that the re-absorption of water, under the removal of over-burden pressure, which may occur in a wet hole, is far less serious than the disturbance which must be caused by the differential pressure acting at the bottom of a deep dry hole tending to force water and soil upward into the casing.

The records of settlements of actual buildings presented in this report illustrate admirably the necessity for thorough soil investigations and intelligent scientific interpretation of boring records, as well as pile and loading tests, in order to avoid bothersome or actually dangerous settlements of important structures.

The problems for research in the field of foundation engineering are practically unlimited, and it is to be hoped that the Committee will continue the fine work which it has sponsored in the past.

T. A. MIDDLEBROOKS,[12] JUN. AM. SOC. C. E. (by letter).[12a]—The Committee, in this report, has made a most valuable contribution to the science of Soil Mechanics. Clearly and concisely it establishes a method of attack in determining the extent of consolidation of clay deposits. The method may not be exact, for reasons suggested in the following discussion, but it will be of untold value in pointing the way to a clearer understanding of the subject.

As stated in the report, the clay mass consolidates as water is squeezed out under an applied load. There is a definite pressure necessary to start the movement of the water through a given thickness of clay, which, for the

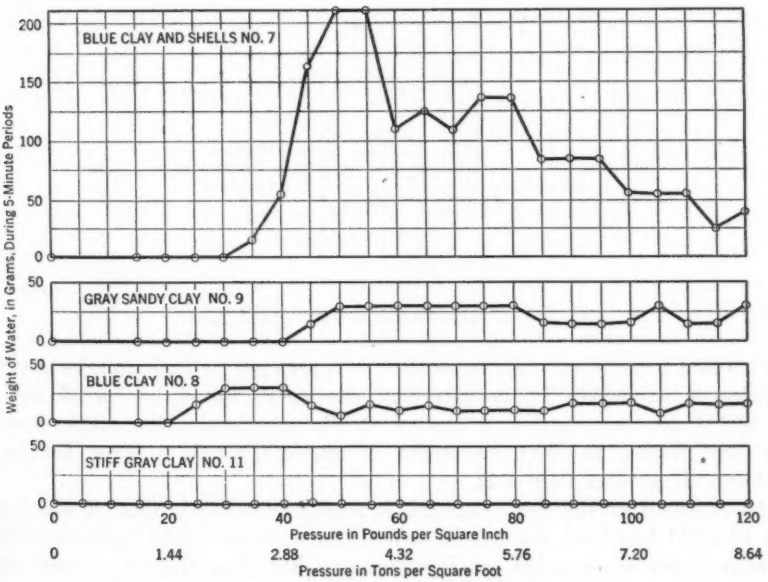

FIG. 51.—PERMEABILITY TESTS ON CLAY DEPOSITS.

lack of a better name, might be termed "back pressure"; and, unless the pressures transmitted to the clay deposit are great enough to overcome this "back pressure" and start the water moving out, no appreciable consolidation will occur. Fig. 51 shows four permeability tests on clay

[12] Chf., Soil Section, U. S. Engr. Office, Vicksburg, Miss.

[12a] Received by the Secretary July 12, 1933.

deposits. The thickness of the sample (see Fig. 52[13]), was 17 in. and it required (as shown), from 1.44 tons per sq ft to more than 8.64 tons per sq ft to start the movement of water through the samples. It is probable that water would always be forced through the thin sample tested in the laboratory if the sample is only 0.02 ft in thickness, as discussed in the report. The load necessary to start percolation of water may be assumed to vary as the thickness of the sample; therefore, it would require only from about 0.02 ton per sq ft to 0.11 ton per sq ft to start the movement of water through a 0.02-ft sample of these clays. From the foregoing and from the fact that

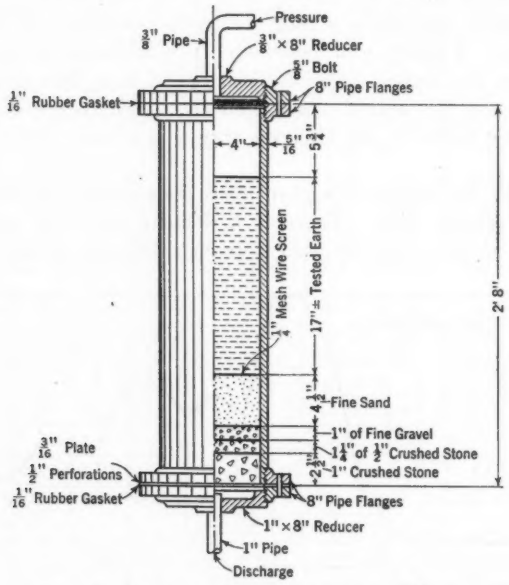

FIG. 52.—APPARATUS FOR PERMEABILITY TESTS.

test loading usually amounts to about 2 or 3 tons per sq ft, it follows that for samples of extremely impervious clay the water would be forced out of the 0.02-ft sample; whereas, in Nature, it might be impossible, within reasonable load limits, to start movement of water through a layer having a thickness no greater than 5 ft.

As to effect of the time element, it might be possible that, if the loads (see Fig. 51) had been applied for several days, water would have percolated through under a somewhat smaller pressure, but Test No. 11 on stiff gray clay shows that there must be some definite pressure that will start movement through a layer of clay since it was run for 2 hours under a pressure from 0 to 8.64 tons per sq ft, without any movement of water. It is scarcely possible that a pressure as low as 2 or 3 tons per sq ft would ever start movement of water through this 17-in. sample.

[13] See, also, Waterworks Handbook, by A. D. Flinn, R. S. Weston, and C. L. Bogert, Members, Am. Soc. C. E., First Edition, 1916, p. 206.

A clay stratum such as that shown in Fig. 14 of the report will not be compressed throughout, since the applied load undoubtedly is not great enough to force water through 57 ft of this clay. The settlement will most likely be due, first, to vertical and horizontal compression of the clay near the surface where the applied load is great enough to force the water out; and, second, to the lateral displacement of the clay from under the load. It is possible that this consolidation will not extend more than 10 or 20 ft below the surface and will probably be compressed in a form similar to the photo-elastic stress diagrams, shown in Figs. 35 to 39, inclusive; and the greater part of the consolidation will most likely be near the surface or the upper limits of the compressible stratum. These conclusions are supported to some extent by the findings of D. P. Krynine, M. Am. Soc. C. E., as reported[14] in 1932.

[14] *Engineering News-Record*, December 29, 1932, pp. 782–784.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from August 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ACCHIONE, LAWRENCE JOSEPH, Philadelphia, Pa. (Age 24.) Refers to H. L. Bowman, S. J. Leonard.

ADAMSON, ARTHUR, New York City. (Age 21.) Refers to C. T. Schwarze, D. S. Trowbridge.

ALEXANDER, MELVILLE RICHMOND, Balboa Heights, Canal Zone. (Age 43.) Asst. Engr., Div. of Office Engr., The Panama Canal. Refers to R. E. Baylor, G. C. Bunker, J. G. Claybourn, B. J. Fletcher, W. B. Godfrey, A. L. Hertz, R. C. Jones, L. B. Moore, E. S. Randolph, J. L. Schley.

ANDERSON, DONALD GASKILL, Chicago, Ill. (Age 22.) Refers to M. L. Enger, N. D. Morgan, C. E. Palmer.

ANDREW, ROBERT MORRISON, Indianapolis, Ind. (Age 25.) Refers to J. T. Hallett, W. K. Hatt, S. C. Hollister, R. B. Wiley.

ANTROBIUS, GEORGE JOHN, East St. Louis, Ill. (Age 25.) Refers to J. J. Doland, G. W. Pickels, T. C. Shedd.

APPLEGATE, DANIEL WEBSTER, Drain, Ore. (Age 26.) Refers to J. R. Griffith, H. S. Rogers.

ARMENTROUT, WILLIAM FRANCIS, Waynesboro, Va. (Age 23.) Refers to R. P. Black, J. H. Johnston, F. C. Snow.

ASHER, VERNON LOMAS, Rolla, Mo. (Age 22.) Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris, C. V. Mann.

ASHFORD, OLIVER JOHN, Seattle, Wash. (Age 24.) Refers to C. W. Harris, G. E. Hawthorn, C. C. More, R. G. Tyler.

ASTON, JAMES WILLIAM, Dallas, Tex. (Age 21.) Refers to J. T. L. McNew, A. P. Rollins.

AURIEMMA, ALFRED ANTHONY, Jersey City, N. J. (Age 23.) Refers to A. Diefendorf, L. C. McCandliss.

AVERY, WILLIAM MURDOCH, Chicago, Ill. (Age 22.) Refers to H. E. Babbitt, G. H. Dell, J. J. Doland, W. C. Huntington, H. H. Jordan, W. H. Rayner, T. C. Shedd.

BAIRD, WALTER DOUGLAS, Dallas, Tex. (Age 24.) Refers to F. R. Naylor, W. R. Spencer.

BALABAN, MORTON LIONEL, Brooklyn, N. Y. (Age 24.) Refers to G. D. Fish, F. E. Foss, G. Morrison.

BALDRIDGE, WILSON DWIGHT, Parma, Idaho. (Age 24.) Refers to J. R. Griffith, H. S. Rogers.

BARBER, CARLTON McCRARY, Wichita, Kans. (Age 28.) Res. Engr., Kansas Highway Comm. Refers to H. Allen, H. D. Barnes, W. V. Buck, L. E. Conrad, E. S. Elcock, W. S. McDaniel, R. B. Wills.

BARCLAY, GEORGE EDWARD, Burley, Idaho. (Age 25.) Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

BARCLAY, ROBERT ELMER, Paterson, N. J. (Age 23.) Refers to H. N. Cummings, W. S. LaLonde, Jr., M. R. Sherrerd.

BARENFANGER, LEWIS EDWARD, Salem, Ill. (Age 24.) County Supt. of Highways, Marion County. Refers to W. W. DeBerard, N. D. Morgan, C. E. Palmer, C. C. Wiley.

BARNHOLT, WILLIAM CHESTER, Mauricetown, N. J. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

BARNUM, VINCENT DUDLEY, Sterling, Ill. (Age 22.) Refers to J. S. Crandell, W. C. Huntington.

BATES, WILLIAM HOWARD, Covington, Ga. (Age 25.) Refers to R. P. Black, F. C. Snow.

BEARD, EARL GILMAN, Jr., Park Ridge, Ill. (Age 21.) Refers to T. L. Condron, G. L. Opper.

BEATTY, RICHARD VILAS, East Liverpool, Ohio. (Age 22.) Refers to C. T. Bishop, J. C. Tracy.

BEEKMAN, HENRY ALEXANDER, New York City. (Age 21.) Refers to C. T. Schwarze, D. S. Trowbridge.

BEHRENS, JOE CHARLES, Chicago, Ill. (Age 22.) Refers to H. E. Babbitt, J. S. Crandell, J. J. Doland, W. A. Oliver, C. C. Wiley.

BELAEFF, NICHOLAS PLATON, Rockville Center, N. Y. (Age 29.) Refers to A. Haring, E. G. Hooper, T. Saville, C. T. Schwarze, D. S. Trowbridge.

BELLAVIA, LOUIS JIACOMO, Garwood, N. J. (Age 24.) Refers to C. D. Billmyer, J. L. Murray.

BENSCHEIDT, ADOLPH, Tillamook, Ore. (Age 23.) Refers to J. R. Griffith, H. S. Rogers.

BLAISDELL, FRED WILLIAM, Goffstown, N. H. (Age 22.) Refers to E. W. Bowler, G. W. Case, J. W. Childs, R. R. Skelton.

BLAKEMAN, HERBERT EDWARD, Brooklyn, N. Y. (Age 20.) Refers to R. E. Goodwin, F. O. X. McLoughlin, J. S. Peck, J. C. Rathbun.

BLANK, WILLIAM ALVIN, Albuquerque, N. Mex. (Age 22.) Refers to J. H. Dorroh, H. C. Neuffer, W. B. Ream.

BLAU, MANUEL, Bronx, N. Y. (Age 29.) With Corbetta Concrete Corporation, New York City. Refers to F. A. Davidson, E. G. Hooper, W. M. Husson, C. T. Schwarze.

BLICKENSDERFER, ALFRED, Cincinnati, Ohio. (Age 25.) Refers to R. E. Hutchins, R. L. McCormick.

BOLEY, ARTHUR LELAND, Sheboygan, Wis. (Age 47.) City Engr. Refers to B. E. Brevik, S. A. Greeley, T. C. Hatton, W. H. Lange, F. A. Torkelson.

BONNETT, CHARLES ALFRED. Eugene, Ore. (Age 26.) Refers to J. R. Griffith, H. S. Rogers.

BONNEY, CHARLES HUTCHINGS, State College, N. Mex. (Age 27.) Refers to H. B. Elmendorf, B. P. Fleming.

BOOHER, LAWRENCE JOSEPH, Tucson, Ariz. (Age 23.) Refers to E. S. Borgquist, F. C. Kelton, J. C. Park.

BOWEN, PHILIP HALE, Ft. Wayne, Ind. (Age 23.) Refers to C. T. Bishop, J. C. Tracy.

BOYD, ALFRED, Grand Forks, N. Dak. (Age 61.) Prof., of Civ. Eng., Univ. of North Dakota. Refers to J. B. Butler, E. F. Chandler, C. Johnson, C. E. Mickey, C. R. Richards, E. R. Stapley, O. V. P. Stout.

BRADY, KENNETH, Houston, Tex. (Age 24.) Refers to L. B. Ryon, Jr., W. E. White.

BRADY, WILLIAM CAMPBELL, State College, N. Mex. (Age 24.) Refers to H. B. Elmendorf, B. P. Fleming.

BRANCH, WILLIAM MARTIN, Augusta, Ga. (Age 24.) Refers to R. P. Black, E. G. Smith, F. C. Snow.

BREWER, JAMES HOBART, George West, Tex. (Age 33.) Asst. Res. Engr., State Highway Dept., Live Oak County, Tex. Refers to T. W. Bailey, F. L. Gartrell, H. C. Hughes, M. Johnson, W. H. Striebeck, Jr.

BRIGHTMAN, EDWIN, Syracuse, N. Y. (Age 21.) Refers to E. F. Berry, S. D. Sarason.

BRINKER, LOUIS EDWIN, Los Angeles, Cal. (Age 25.) Refers to R. M. Fox, D. M. Wilson.

BRODAHL, JACK ASBJORN AUME, El Monte, Cal. (Age 31.) Field Engr. with Clinton Constr. Co., Los Angeles, Cal. Refers to N. Bostwick, N. Charde, T. E. Ferneau, F. F. Friend, M. Graham, Jr., G. A. Greene, R. L. Gregory, C. W. Pierce, F. Vogt.

BROWN, ELLIS ROBERT, Ashland, Mass. (Age 20.) Refers to J. W. Howe, A. J. Knight.

BRUDENELL, ROSS NYMAN, Iowa City, Iowa. (Age 22.) Refers to F. A. Nagler, D. L. Yarnell.

BUFORD, JACK WILLIAM, Seattle, Wash. (Age 21.) Refers to E. B. Crane, G. E. Hawthorn, C. C. May, R. J. Middleton, C. C. More, R. G. Tyler.

BUNDY, WILLARD LINDLEY, Tacoma, Ohio. (Age 27.) Refers to G. E. Large, C. T. Morris.

BURKHARDT, SCOTT, St. Helens, Ore. (Age 23.) Refers to J. R. Griffith, C. A. Mockmore, H. S. Rogers.

BURKO, WILLIAM, New York City. (Age 22.) Refers to H. P. Hammond, E. J. Squire.

BUTLER, VAN BROUGH, Seattle, Wash. (Age 24.) Refers to G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

CALHOUN, FRANK ELBERT, Pensacola, Fla. (Age 23.) Refers to W. T. Lyle, W. E. Wheat.

CAPLAN, DAVID, Brooklyn, N. Y. (Age 23.) Eng Asst., Dept. of Plant and Structures, New York City. Refers to F. E. Foss, G. Morrison.

CAPLING, LEONARD, Maywood, Ill. (Age 25.) Refers to H. C. Bird, W. H. Hall.

CARBONE, BARTHOLOMEW LEON, Brooklyn, N. Y. (Age 26.) Refers to E. R. Cary, L. W. Clark.

CARR, GILES LACEY, Chandlerville, Ill. (Age 25.) Refers to E. E. Bauer, J. S. Crandell, W. C. Huntington, G. W. Pickels, T. C. Shedd, C. C. Wiley.

CARSTENSEN, HERMAN CHARLES, Bowling Green, Ohio. (Age 33.) Bridge Engr., Wood County. Refers to E. S. Fenelon, G. B. Gascoigne, T. S. Johnson, H. P. Jones, N. W. Locke, J. B. Morrison, B. H. Prack, R. C. Reese, R. M. Strohl.

CARTER, ELDEN WUEST, Portland, Ore. (Age 22.) Refers to J. R. Griffith, J. P. Newell, H. S. Rogers, J. C. Stevens.

CARTER, GEORGE RICHMOND, Danville, Ill. (Age 25.) Refers to W. C. Huntington, T. C. Shedd.

CASEY, HUGH JOHN, Berlin, Germany. (Age 35.) Freeman Traveling Scholarship (Am. Soc. Mech. Engrs.). Refers to L. Brown, E. L. Daley, H. Deakyne, C. W. Kutz, W. H. McAlpine, E. M. Markham, G. B. Pillsbury, B. B. Somervell.

CASTELLAN, NORMAN JOHN, Denver, Colo. (Age 21.) Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder.

CASTELLUCCI, JOSEPH, Providence, R. I. (Age 22.) Refers to C. D. Billmyer, J. L. Murray.

CHANG, HAN YING, Tientsin, China. (Age 33.) Senior Engr., Comm. for Development of Great Northern Port. Refers to F. A. Barnes, L. S. Chan, H. Cross, S-T. Li, H. H. Ling, E. W. Schoder.

CLARK, ANDREW ELDON, Albuquerque, N. Mex. (Age 27.) Refers to H. B. Elmendorf, B. P. Fleming.

CLICKENER, CORWIN KING, Waukegan, Ill. (Age 23.) Refers to F. R. McMillan, G. A. Maney.

CLINTON, ROBERT, Braintree, Mass. (Age 28.) Refers to L. W. Clark, T. R. Lawson.

CLOSE, HERBERT SCHOCK, Plainfield, N. J. (Age 22.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

CLOUDMAN, CHARLES GREENLEAF, Kingston, R. I. (Age 27.) Refers to C. D. Billmyer, J. L. Murray.

COHEN, DANIEL, Chicago, Ill. (Age 22.) Refers to H. E. Babbitt, J. J. Doland, W. C. Huntington, G. W. Pickels, T. C. Shedd.

COMERFORD, WILLIAM PIERRE, New York City. (Age 22.) Refers to R. E. Goodwin, M. Henry, F. O. X. McLoughlin, J. S. Peck, H. J. Plock, J. C. Rathbun.

COMINS, HARRISON DURGIN, Vineland, N. J. (Age 24.) Refers to M. O. Fuller, H. G. Payrow.

COPLIN, PHILIP LOUIS, Albuquerque, N. Mex. (Age 24.) Refers to J. H. Dorroh, H. C. Neuffer.

CORSAN, BENJAMIN, Bronx, N. Y. (Age 22.) Refers to F. E. Foss, G. Morrison.

COTTON, JOHN CASKEY, Bethesda, Md. (Age 23.) Refers to E. R. Cary, L. W. Clark, T. R. Lawson.

CRAWFORD, EDGAR GRENFELLE, Pasadena, Cal. (Age 21.) Refers to R. R. Martel, W. W. Michael, F. Thomas.

CRAWFORD, GLENN ARTHUR, Covina, Cal. (Age 24.) City Engr. Refers to E. S. Borgquist, F. C. Kelton, J. C. Park.

CROLL, DONALD, Buffalo, N. Y. (Age 22.) Refers to C. T. Bishop, J. C. Tracy.

CROPPER, GEORGE BERTRAND, Ocean City, Md. (Age 25.) Refers to H. C. Bird, W. H. Hall.

CROWE, GEORGE FREDERICK, Seattle, Wash. (Age 25.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

DABB, ALBERT HENRY, Jr., Linden, N. J. (Age 22.) Refers to H. N. Cummings, W. S. LaLonde, Jr., J. Verner.

DACK, BRUCE M, Los Angeles, Cal. (Age 27.) Refers to F. J. Converse, R. R. Martel, W. W. Michael, F. Thomas.

DANDRIDGE, WILLIAM EATON, Hoboken, N. J. (Age 21.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

DARLING, HORACE VELPEAU, Washington, D. C. (Age 30.) Associate Engr., U. S. Govt. Refers to J. R. Lapham, S. B. Williamson.

DASH, WILLIAM RODNEY, Brooklyn, N. Y. (Age 21.) Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

DAVIDSON, ARTHUR JEROME, Moscow, Idaho. (Age 23.) Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

DAVIES, NORMAN OWEN, Seattle, Wash. (Age 22.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

DAVISON, MILBERN HARRY, Kansas City, Kans. (Age 21.) Engr., Min. Dept., Central Coal & Coke Co., Kansas City, Mo. Refers to L. E. Conrad, E. R. Dawley, F. F. Frazier, M. W. Furr, C. H. Scholer.

DeHART, STANLEY BURSON, Sheepshead Bay, N. Y. (Age 27.) Refers to H. P. Hammond, W. K. Hatt, G. E. Lommel, R. Ridgway, E. J. Squire, H. D. Winsor.

de JONG, JOHN, Troutdale, Ore. (Age 28.) With Oregon State Highway Dept. Refers to J. R. Griffith, H. S. Rogers.

DEMERITT, GEORGE WESLEY, Elmhurst, N. Y. (Age 21.) Refers to C. T. Schwarze, D. S. Trowbridge.

DE MILITA, JOSEPH, Los Angeles, Cal. (Age 23.) Refers to F. J. Converse, W. W. Michael, F. Thomas.

DER, JACK HIM, Hong Kong, China. (Age 25.) Refers to H. Bouchard, W. J. Emmons, R. L. Morrison, W. C. Sadler, R. S. Swinton, J. S. Worley.

DeVOE, STEPHEN JOSEPH, Jr., Woodside, N. Y. (Age 25.) Refers to F. A. Biberstein, Jr., J. J. Costa, R. A. O'Hara, A. V. Sheridan.

DIETZ, OLEN ADOLPH, Powell, Miss. (Age 26.) Inspector, U. S. Engr. Dept., Helena, Ark. Refers to H. K. Gatley, G. R. Kavanagh, R. McNicholas, K. W. Markwell, L. W. Mashburn, H. V. Pittman, C. H. Schwartz, B. B. Somervell.

DIGENNARO, JOHN ANTHONY, Bayonne, N. J. (Age 21.) Refers to C. Steiner, J. J. Sweeney.

DI GIACINTO, ALBERT GEORGE, Elmsford, N. Y. (Age 22.) Refers to H. E. Breed, A. Haring, E. G. Hooper, T. Saville, C. T. Schwarze, D. S. Trowbridge.

DOYLE, WILLIAM CHRISTIAN, Seattle, Wash. (Age 21.) Refers to C. W. Harris, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

DUNBAR, T J, Jr., Memphis, Tex. (Age 24.) Refers to E. C. H. Bantel, J. A. Focht, C. L. Hasie, T. U. Taylor.

EARDLEY, JAMES JOHN, Bronx, N Y. (Age 22.) Refers to J. J. Costa, A. V. Sheridan.

ECKELMEYER, ALBERT FREDERICK, Philadelphia, Pa. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

EHASZ, FRANCIS LOUIS, Long Island City, N. Y. (Age 22.) Refers to H. E. Breed, E. G. Hooper, T. Saville, C. T. Schwarze, D. S. Trowbridge.

ELLIOT, DONALD GEORGE, Montreal, Que., Canada. (Age 23.) Designer with Monsarrat & Pratley. Refers to C. N. Monsarrat, J. W. Roland.

ELLIOTT, ARTHUR LEONARD, Portland, Ore. (Age 21.) Refers to J. A. Elliott, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler, R. B. Wright.

ELLIS, GENE, Council Grove, Kans. (Age 22.) Refers to L. E. Conrad, F. F. Frazier, M. W. Furr, C. H. Scholer, L. V. White.

ENANDER, CARL ALFRED, Dunellen, N. J. (Age 26.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

ERICKSON, HARRY WILBERT, Seattle, Wash. (Age 22.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

FAHRBERGER, LOUIS FRANK, Jr., Chicago, Ill. (Age 20.) Refers to J. G. Bennett, C. B. Burdick, T. L. Condron, A. J. Hammond, W. E. Hart.

FALLON, ROBERT FORSYTH, Buffalo, N. Y. (Age 20.) Refers to F. A. Barnes, J. E. Perry, R. Y. Thatcher, P. H. Underwood.

FEICHTMEIR, ARMAND CASIMIR, Wausau, Wis. (Age 23.) Refers to E. L. Grant, C. Moser, L. B. Reynolds, E. C. Thomas.

FERBRACHE, CHARLES PHILIP, Rolla, Mo. (Age 26.) Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton.

FERNALD, GORDON HILDRETH, Jr., San Bernardino, Cal. (Age 23.) Refers to G. H. Fernald, E. L. Grant, L. B. Reynolds, J. B. Wells, H. A. Williams.

FITZPATRICK, JOSEPH ALOYSIUS, New York City. (Age 29.) Refers to E. B. Fitzpatrick, F. E. Foss.

FLINT, JOHN BERNARD, Phoenix, Ariz. (Age 20.) Refers to A. H. Harter, F. C. Kelton, L. C. Lashmet, T. Maddock.

FLUCK, MILES DIETRICH, Telford, Pa. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

FOGARTY, FRANCIS WILLIAM LAWRENCE, West Haven, Conn. (Age 21.) Refers to C. T. Bishop, J. C. Tracy.

FOSTER, WENDELL LEE, Oklahoma City, Okla. (Age 24.) Refers to J. E. Kirkham, E. R. Stapley.

GABRIEL, JOHN MACK, Garfield, N. J. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

GARBARINI, EDGAR JOSEPH, Oakland, Cal. (Age 23.) Refers to C. Derleth, Jr., B. A. Etcheverry, F. S. Foote, S. T. Harding, C. G. Hyde, B. Jameyson.

GARD, WALTER SUMNER, Charleston, W. Va. (Age 22.) Refers to J. L. Newcomb, L. H. Williamson.

GARDINER, ARTHUR PERRY, Seattle, Wash. (Age 22.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G Tyler.

GARDNER, LEIGH OWEN, Miami, Ariz. (Age 20.) Refers to E. S. Borgquist, F. C. Kelton, J. C. Park.

GARDNER, ROBERT ANTHONY, South Gibson, Pa. (Age 26.) Instructor in Civ. Eng., Bucknell Univ., Lewisburg, Pa. Refers to D. M. Griffith, W. C. Hoad, R. O'Donnell, H. B. Shattuck, E. D. Walker.

GELARDI, ANTHONY, Providence, R. I. (Age 22.) Refers to C. D. Billmyer, J. L. Murray.

GEYER, ROBERT CARL, Chicago, Ill. (Age 21.) Refers to W. C. Huntington, N. D. Morgan.

GILBERT, GEORGE DOUGLAS, Sacramento, Cal. (Age 25.) Refers to F. L. Bixby, H. P. Boardman.

GLASHEEN, WILLIAM JOHN, New Rochelle, N. Y. (Age 23.) Refers to J. J. Costa, F. C. Zeigler.

GREGORY, THOMAS McFARLAND, Dallas, Tex. (Age 25.) Batcherman, W. H. Nichols & Co. Refers to J. H. Knox, A. J. McKenzie, T. A. Munson.

GRIFFITHS, JOHN DAVID, Cincinnati, Ohio. (Age 24.) Refers to R. A. Anderegg, H. H. Kranz.

GROTE, ROBERT WALLACE, Denver, Colo. (Aged 20.) Refers to B. A. Etcheverry, S. T. Harding, W. L. Prouty.

GROVER, LaMOTTE, Topeka, Kans. (Age 32.) Bridge Engr. (Office), Dept. of Design, Kansas State Highway Comm. Refers to W. V. Buck, L. E. Conrad, O. J. Eidmann, F. W. Epps, H. A. Marshall, J. I. Quinn, C. H. Scholer.

GUGLIELMONE, GEORGE ALEXANDER, Bronx, N. Y. (Age 20.) Refers to H. P. Hammond, L. F. Rader.

GUYN, THOMAS CORWIN, Ft. Hall, Idaho. (Age 47.) Project Engr., U. S. Indian Irrigation Service. Refers to L. M. Holt, L. T. Jessup, M. A. Newell, F. H. Pickett, P. J. Preston, W. M. Reed.

HAINES, DONALD HUTCHINSON, Seattle, Wash. (Age 23.) Refers to G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

HALDEMAN, EDWARD GEORGE, East Orange, N. J. (Age 21.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

HALL, GEORGE YAYOUNG, San Francisco, Cal. (Age 23.) Refers to R. E. Davis, C. Derleth, Jr., C. G. Hyde, B. Jameyson, G. E. Troxell.

HALL, NORMAN HENRY, Des Moines, Iowa. (Age 25.) Refers to J. F. Brookes, N. E. Wolfard.

HANKINS, LAWRENCE DONALD, Coeur d'Alene, Idaho. (Age 25.) Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

HANSEN, RALPH CHRISTIAN, Seattle, Wash. (Age 22.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

HARDER, ARTHUR WILLIAM, Buffalo, N. Y. (Age 23.) Refers to E. F. Berry, L. Mitchell.

HARDING, ROBERT CARNEGIE, Tucson, Ariz. (Age 25.) Refers to F. C. Kelton, H. C. Schwalen, G. E. P. Smith.

HARRIS, GENE LEWIS, Joplin, Mo. (Age 25.) Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

HARRIS, ROBERT MICHAEL SYDNEY, Payette, Idaho. (Age 21.) Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

HAUGER, ROY LEO, San Antonio, Tex. (Age 22.) Refers to J. T. L. McNew, J. J. Richey.

HAWES, LORIN LINDLEY, Coya, Rancagua, Chile. (Age 31.) Asst. Hydr. Engr., Braden Copper Co. Refers to C. D. Ashbrook, C. Barahona Munoz, P. L. Spanne, H. D. Stockwell, J. W. Straney.

HAYNES, WESLEY EATON, Concord, N. H. (Age 22.) Jun. Engr. and Draftsman, New Hampshire Highway Dept. Refers to E. W. Bowler, M. W. Messer, R. R. Skelton.

HEINZ, OTTO, Jr., Philadelphia, Pa. (Age 25.) Refers to H. L. Bowman, S. J. Leonard.

HELDENFELS, FREDERICK WILLIAM, Jr., Beeville, Tex. (Age 21.) Refers to L. E. Grinter, J. T. L. McNew, J. J. Richey.

HELLMICH, WALTER FREDERICK, St. Louis, Mo. (Age 24.) Refers to N. D. Morgan, C. E. Palmer.

HERMAN, BRUCE ALFRED, Baltimore, Md. (Age 21.) Refers to J. H. Gregory, J. T. Thompson.

HICKS, GARDNER WORDELL, Tiverton, R. I. (Age 21.) Refers to T. R. Camp, E. Mirabelli, C. M. Spofford.

HIEGL, GEORGE, Jr., Philadelphia, Pa. (Age 24.) Refers to H. L. Bowman, S. J. Leonard.

HILL, ARCHIE KERR, Logan, Utah. (Age 23.) Refers to G. D. Clyde, B. A. Etcheverry, S. T. Harding, O. W. Israelsen, H. R. Kepner.

HILL, JACOB ERNEST, Brush Prairie, Wash. (Age 23.) Refers to H. E. Phelps, M. K. Snyder, J. G. Woodburn.

HOFFMAN, IRVING CHARLES, West Los Angeles, Cal. (Age 28.) Engr. Los Angeles County Regional Planning Comm. Refers to R. M. Fox, W. J. Fox, A. L. Sonderegger, A. S. Zinn.

HOLZMAN, JOHN, Brooklyn, N. Y. (Age 21.) Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

HOOPER, HUGH BRAXTON, Chattanooga, Tenn. (Age 45.) Asst. to Dist. Engr., U. S. Engr. Dept. Refers to C. A. Betts, H. C. Fiske, F. C. French, W. R. King, O. J. Miller, A. F. Porzelius, E. E. Teeter, A. H. Van Cleve, D. B. Ventres, J. Wright.

HUBER, LEVI ALBERTSON, Bridgeton, N. J. (Age 22.) Refers to W. H. Barton, Jr., C. E. Myers.

HUGHES, LEATON LEWIS, St. Louis, Mo. (Age 27.) Refers to E. O. Sweetser, J. L. Van Ornum.

HUHS, ROY ELBERT, Berkeley, Cal. (Age 26.) Locating Engr., U. S. Forest Service. Refers to R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, C. G. Hyde, C. L. Young.

HUMES, JOHN DONALD, Seattle, Wash. (Age 27.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

HUNT, THEODORE WILLIAM, East Greenbush, N. Y. (Age 24.) Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

HURST, RUSSEL FRANCIS, Jr., West Chester, Pa. (Age 20.) Refers to C. A. Howland, J. J. Sweeney, I. S. Walker.

HUSSEY, PIERCE LEAPSON, Berkeley, Cal. (Age 23.) Refers to B. A. Etcheverry, S. T. Harding.

IRVINE, JAMES OSBORNE, Jersey City, N. J. (Age 29.) Salesman, National Cash Register Co. Refers to M. A. Butler, H. R. Gabriel, C. Gilman, N. H. Jones, W. H. Robertson, C. H. Splitstone.

IRWIN, FREDRICK BRUCE, Upper Sandusky, Ohio. (Age 22.) Refers to G. H. Elbin, A. R. Webb.

IZATT, JOHN ORMOND, Grace, Idaho. (Age 24.) Rodman, U. S. Coast and Geodetic Survey. Refers to I. N. Carter, I. C. Crawford, R. M. Harris, J. W. Howard.

JAKUBOWSKY, JOSEPH FELIX JOHN, Brooklyn, N. Y. (Age 23.) Civ. Engr. at Ft. Hamilton, N. Y. Refers to L. W. Clark, H. B. Compton, T. R. Lawson, H. O. Sharp.

JANSSEN, ALLEN SHEELEY, Moscow, Idaho. (Age 25.) Instructor, Civ. Eng. Dept., Univ. of Idaho. Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

JAYNE, STEPHEN CORNELIUS, Wenatchee, Wash. (Age 27.) Res. Engr., State Highway Dept. of Washington. Refers to O. R. Elwell, C. W. Nash, H. E. Phelps, H. A. Sewell, R. K. Tiffany.

JENKINS, JOHN JOSEPH, Jr., Baltimore, Md. (Age 22.) Refers to J. H. Gregory, J. T. Thompson.

JENSEN, VERNON PETER, Ames, Iowa. (Age 29.) Asst. Prof., Dept. of T. & A. M., Iowa State Coll. Refers to H. Cross, M. L. Enger, H. J. Gilkey, W. C. Huntington, A. Marston, F. E. Richart, H. M. Westergaard.

JOHNSON, ALBERT EDWIN, San Bernardino, Cal. (Age 23.) Refers to J. R. Griffith, C. A. Mockmore, H. S. Rogers.

JOHNSON, CHARLES JAMES, Portland, Ore. (Age 26.) Refers to J. W. Cunningham, J. R. Griffith, G. W. Holcomb, R. E. Koon, C. A. Mockmore, H. S. Rogers, D. H. Rowe, J. C. Stevens.

JOHNSON, JOHN WILLIAM, Pensauken, N. J. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

JOHNSON, KENNETH GORDON, Seattle, Wash. (Age 23.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

JONES, EDWIN ALFRED, Wenatchee, Wash. (Age 24.) Refers to I. L. Collier, G. E. Hawthorn, C. C. More, R. G. Tyler.

JOSLYN, CHARLES SHEAFE, New York City. (Age 35.) Civ. Engr., Marc Eidlitz & Son. Refers to F. N. Benedict, P. D. G. Hamilton, A. F. Holler, T. H. Jackson, A. H. Jorgensen, T. T. Knappen, T. R. Lawson.

JOY, DONALD GORDON, Jamaica Estates, N. Y. (Age 22.) Refers to P. G. Laurson, J. C. Tracy.

JOYCE, JOSEPH PETER, Jr., Chicago Ill. (Age 22.) Refers to J. S. Crandell, J. J. Doland.

KANE, JOHN EDMUND, Elizabeth, N. J. (Age 28.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

KENNEY, MILTON TURNER, Lowell, Ind. (Age 26.) Refers to W. K. Hatt, W. J. Henderson, S. C. Hollister, W. E. Howland, G. E. Lommel, R. B. Wiley.

KETCHUM, SMITH ADRON, Canon City, Colo. (Age 25.) Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder.

KIMMEL, ALFRED STEPHEN, Miamisburg, Ohio. (Age 22.) Refers to J. J. Chamberlain, Jr., B. T. Schad.

KING, JOHN CALLEN, Jr., Westwood, N. J. (Age 22.) Refers to J. B. Babcock, 3d, W. M. Fife, C. M. Spofford.

KIRKPATRICK, GEORGE ALEXANDER, Portland, Ore. (Age 23.) Refers to G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

KITCH, RICHARD BENSON, Oak Park, Ill. (Age 23.) Refers to H. Cross, W. C. Huntington, C. E. Palmer.

KLINE, ARTHUR, Barkhamsted, Conn. (Age 47.) Div. Engr., Metropolitan Dist. Water Bureau, Hartford, Conn. Refers to H. R. Buck, H. W. Griswold, W. E. Johnson, C. M. Saville, W. L. Voorduin.

KOENIG, JOHN WILLIAM, Cincinnati, Ohio. (Age 27.) Refers to G. E. Large, C. T. Morris.

KOPP, ALVIN HENRY, Newark, N. J. (Age 21.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

KREISS, ROBERT FREDERICK, Minneapolis, Minn. (Age 23.) Refers to F. Bass, A. S. Cutler.

KRUSE, MARVIN OTTO, Hartley, Iowa. (Age 25.) Refers to T. R. Agg, R. A. Caughey, J. S. Dodds, A. H. Fuller, W. E. Galligan, F. Kerekes, R. A. Moyer, L. O. Stewart.

LANGMAN, RICHARD SHERRARD, Rock Island, Ill. (Age 21.) Refers to H. E. Babbitt, H. Cross, W. C. Huntington.

LARKIN, KENNETH HOYT, Kansas City, Mo. (Age 25.) Draftsman and Jun. Engr., Butler Mfg. Co. Refers to W. C. McNown, H. A. Rice.

LARROWE, ALBERTUS EUGENE, Portland, Ore. (Age 22.) Refers to J. R. Griffith, H. S. Rogers.

LEIGHTON, CHARLES KELTON, Corsicana, Tex. (Age 26.) Refers to M. L. Bowers, W. W. McClendon, J. T. L. McNew, J. J. Richey.

LEITHMANN, WARREN JUSTIN, Jr., Philadelphia, Pa. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

LeMOYNE, CHARLES, Jr., Boulder City, Nevada. (Age 24.) Refers to I. N. Carter, J. P. Congdon, I. C. Crawford, J. W. Howard, R. J. Newell.

LENHART, JACK, Minneapolis, Minn. (Age 24.) Refers to F. Bass, A. S. Cutler.

LeROY, JAMES MILTON, Elizabeth City, N. C. (Age 22.) Refers to C. L. Mann, W. C. Riddick, H. Tucker, J. S. Whitener.

LEWIS, PERLEY MITCHELL, Phoenix, Ariz. (Age 34.) Lewis Bros. Engrs. and Contrs. Refers to R. G. Baker, C. C. Cragin, I. C. Crawford, V. H. Housholder, F. J. O'Hara.

LEWIS, RICHARD GASTON, Hopewell, Va. (Age 24.) Refers to R. B. H. Begg, F. J. Sette.

LICCIONE, DANIEL JOSEPH, New York City. (Age 27.) Refers to H. P. Hammond, L. F. Rader, M. A. Rifkinson, E. J. Squire.

LIN, TUNG YEN, Berkeley, Cal. (Age 21.) Refers to R. E. Davis, C. Derleth, Jr., C. G. Hyde, B. Jameyson.

LOUREE, CHARLES HAROLD, Tulsa, Okla. (Age 30.) With Tulsa County Highway Dept. Refers to V. H. Cochrane, W. E. Davis, R. L. Ehrlich, R. P. Garrett, A. Lewald, J. B. Neely, D. W. Patton, C. Schultz.

McALEER, JOHN BERNARD, North Weymouth, Mass. (Age 22.) Refers to J. B. Babcock, 3d, C. B. Breed, C. M. Spofford.

McGAUGHEY, PAUL THOMAS, Philadelphia, Pa. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

McGILVRA, ROBERT DICKINSON, Portland, Ore. (Age 22.) Refers to J. B. Babcock, 3d, C. B. Breed, C. M. Spofford.

McGOWAN, THOMAS FRANCIS, San Francisco, Cal. (Age 23.) Rodman. Div. of Highways, Sacremento, Cal. Refers to C. Derleth, Jr., C. G. Hyde.

McLAUGHLIN, CHILTON WHITE, Jr. Kansas City, Kans. (Age 23.) County Surveyor, Wyandotte County, Kans. Refers to L. Arnold, G. W. Bradshaw, J. O. Jones, W. C. McNown, H. A. Rice.

McNAMARA, STEPHEN JAMES, Prestwich, Man., England. (Age 34.) Asst. Chf. Estimating and Designing Engr., Grip Steel

Bar Co., Ltd. Refers to H. W. Ballou, R. W. Burpee, S. C. Jemian, E. M. Viner. Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

MAHLER, LOUIS VICTOR, Philadelphia, Pa. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

MAHON, JUSTIN DAVID, Jersey City, N. J. (Age 24.) Refers to A. H. Beyer, D. M. Burmister, J. K. Finch, W. J. Krefeld, C. R. Wyckoff.

MAINARDI, POMPEY, Paterson, N. J. (Age 22.) Refers to H. N. Cummings, W. S. LaLonde, Jr., M. R. Sherrerd.

MAKLE, ALBERT LEONARD, Palo Alto, Cal. (Age 24.) Refers to J. C. L. Fish, C. Moser, L. B. Reynolds, E. C. Thomas, J. B. Wells.

MALE, SEYMOUR, New York City. (Age 24.) Refers to C. T. Schwarze, D. S. Trowbridge.

MALONE, JAMES ROBERT, Durham, N. C. (Age 24.) Refers to H. C. Bird, W. H. Hall.

MANNING, FRANK GERARD, Jersey City, N. J. (Age 22.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

MARSH, EDWARD THEODORE, Philadelphia, Pa. (Age 24.) Refers to H. L. Bowman, S. J. Leonard.

MARTA, GEORGE ARTHUR, Philadelphia, Pa. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

MAST, FREDERICK WILLIAM, Quincy, Ill. (Age 23.) Refers to N. D. Morgan, C. E. Palmer.

MATHOFF, DAVID, Peabody, Mass. (Age 39.) Asst. to Chf. Engr. and Structural Designer, Grid Flat Slab Corporation, Dorchester, Mass. Refers to E. F. Allbright, S. M. Brown, O. H. Horovitz, M. Linenthal, E. A. Norwood.

MAYER, SAM SOMA, New York City. (Age 27.) Refers to F. E. Foss, G. Morrison, J. P. J. Williams.

MAYES, PHILLIP, Seattle, Wash. (Age 23.) Refers to O. P. M. Goss, C. W. Harris, G. E. Hawthorn, C. C. May, J. W. Miller, C. C. More, A. H. Riney.

MIDDLETON, ROBERT ALFRED, Seattle, Wash. (Age 22.) Refers to I. L. Collier, E. B. Crane, G. E. Hawthorn, C. C.. May, C. C. More, R. G. Tyler.

MILENS, CYRIL AARON, Kansas City, Mo. (Age 22.) Refers to H. E. Babbitt, J. J. Doland, W. A. Oliver.

MILES, WALTER FRANK, Tacoma, Wash. (Age 26.) Engr., State Coll. of Washington, Pullman, Wash. Refers to H. E. Phelps, M. K. Snyder, J. G. Woodburn.

MILLER, ROBERT SMITH, Oxford, Ohio. (Age 23.) Refers to H. C. Bird, W. H. Hall.

MITCHELL, PARK, Jr., Philadelphia, Pa. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

MOCHAMER, THOMAS RICHARD, Centralia, Pa. (Age 22.) Refers to W. H. Barton, Jr., C. E. Myers.

MODAK, BHASKAR LAXMAN, Indore, Central India. (Age 48.) Executive Engr., Govt. of Bombay, Water Supply and Drainage Div. Refers to S. K. Gurtu, C. R. Katkoria.

MOHL, EMANUEL ISRAEL, Rochelle Park, N. J. (Age 23.) Refers to A. Haertlein, L. J. Johnson, H. M. Turner.

MONACO, JOHN CHARLES, Kearney, N. J. (Age 22.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

MOORE, FRANCIS COCHRANE, Amherst, Mass. (Age 21.) Refers to J. W. Howe, C. F. Meyer, J. E. Ostrander.

MORITZ, GORDON DURRIE, Bound Brook, N. J. (Age 23.) Refers to G. J. Davis, Jr., A. C. Decker, D. C. A. duPlantier, S. C. Houser.

MORLAND, KENNETH DeFORD, Berwyn, Ill. (Age 21.) Refers to H. E. Babbitt, J. J. Doland, G. W. Pickels, T. C. Shedd, F. W. Stubbs, Jr., J. Vawter, C. C. Wiley.

MORRIS, EDGAR LONGETTE, Washington, D. C. (Age 22.) Refers to E. L. Clarke, H. E. Glenn.

MORRIS, JOHN HITE, Morgantown, W. Va. (Age 22.) Refers to G. P. Boomsliter, L. V. Carpenter, H. O. Cole, R. P. Davis, W. S. Downs.

MORTENSEN, CLIFFORD NELSE, Seattle, Wash. (Age 21.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

MORTON, CHARLES MOREHEAD, San Marcos, Tex. (Age 25.) With Robt. E. McKee, Gen. Contr. Refers to P. M. Ferguson, S. P. Finch, G. W. Gill, T. U. Taylor.

MURPHY, EDWARD EYRE, Buffalo, N. Y. (Age 29.) Refers to G. J. Davis, Jr., D. C. A. duPlantier, S. C. Houser.

MURPHY, EDWARD JOSEPH, Bronx, N. Y. (Age 23.) Refers to J. J. Costa, A. V. Sheridan.

NARDELLI, DANTE, Providence, R. I. (Age 24.) Refers to C. D. Billmyer, J. L. Murray.

NEIDER, RUSSELL CHARLES, Hillside, N. J. (Age 22.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

NEWBERGER, REUBEN, Newark, N. J. (Age 21.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

NEWHALL, GEORGE NELSON, Berkeley, Cal. (Age 22.) Refers to C. Derleth, Jr., B. A. Etcheverry, C. G. Hyde.

NEWMAN, EDWIN MAHLON, Kansas City, Mo. (Age 24.) Refers to L. E. Conrad, H. E. Durham, F. F. Frazier, M. W. Furr.

NIXON, ANGUS GRAY, Canton, Tex. (Age 23.) With Texas Highway Dept. (Refers to L. E. Grinter, J. T. L. McNew, T. A. Munson, J. J. Richey, C. E. Sandstedt.

NOBES, CHARLES WILLIAM, Frankfort, Ind. (Age 22.) Refers to J. J. Doland, W. A. Oliver.

NUDENBERG, NATHANIEL JOHN, Newark, N. J. (Age 21.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

NUTE, JOHN WARREN, Los Altos, Cal. (Age 25.) Refers to J. C. L. Fish, E. L. Grant, L. B. Reynolds.

O'BRIEN, FRANCIS XAVIER, West Englewood, N. J. (Age 23.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

OCHS, MALCOM GEORGE, San Jose, Cal. (Age 31.) Head Estimator with W. J. Ochs, Gen. Contr. Refers to W. S. Kingsbury, Jr., S. P. Laverty, J. G. McMillan, L. Pope, 3d, E. L. Soule.

O'DONNELL, WILLIAM JOSEPH, South Ozone Park, N. Y. (Age 22.) Refers to F. E. Foss, G. Morrison, L. C. Urquhart, J. P. J. Williams.

OGDEN, HORATIO NASH, New Orleans, La. (Age 24.) Refers to F. C. Carey, D. Derickson, W. B. Gregory, H. D. Moore, J. Riess.

OGLE, WILLIAM WARREN, Greenwich, Conn. (Age 25.) Refers to L. W. Clark, W. R. Devaul.

OMAN, ARLIE ANDERS, Napoleon, N. Dak. (Age 23.) Refers to R. E. Kennedy, W. E. Smith.

OMSTED, HARALD, Pasadena, Cal. (Age 32.) Refers to R. C. Booth, R. R. Martel, R. W. Spencer, F. Thomas, I. L. Tyler, M. D. Williams.

ONDRUS, THOMAS, Berwyn, Ill. (Age 21.) Refers to H. E. Babbitt, W. C. Huntington, G. W. Pickels.

ORIVE ALBA, ADOLFO, Mexico City, Mexico. (Age 27.) Chf. Engr., Mexican Irrigation Comm. (Mexican Reclamation Service). Refers to H. P. Bunger, C. H. Howell, S. W. Stewart, A. Weiss, C. P. Williams.

OTTO, ARTHUR LOUIS, Newark, N. J. (Age 23.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

PAGE, OTIS MOORE, Memphis, Tenn. (Age 45.) With U. S. Dept. of Agriculture, Bureau of Eng. Refers to C. E. Boesch, G. R. Boyd, G. C. Dobson, J. M. Johnson, L. A. Jones, S. H. McCrory, B. B. Somervell.

PALM, ALF RAYMOND, Chicago, Ill. (Age 21.) Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder, W. H. Thoman.

PAPANDREA, NATALE NEO, Newark, N. J. (Age 25.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

PARKER, ANTOINE PANET, Flushing, N. Y. (Age 22.) Refers to C. T. Bishop, J. C. Tracy.

PARKS, RECTOR, Pleasant Lake, Ind. (Age 25.) Refers to W. K. Hatt, S. C. Hollister, R. B. Wiley.

PARSONS, CHESTER NIMITZ, Comstock, Tex. (Age 25.) Refers to M. B. Hodges, G. M. Jowers, H. P. Stockton, Jr.

PARSONS, NORMAN, WELBEC, Morgantown, W. Va. (Age 22.) Refers to G. P. Boomsliter, L. V. Carpenter, R. P. Davis, W. S. Downs.

PELTIER, EUGENE JOSEPH, Concordia, Kans. (Age 23.) Refers to L. E. Conrad, F. W. Epps, F. F. Frazier, M. W. Furr, C. H. Scholer, L. V. White.

PENNER, FRANK JOSEPH, Jr., Brooklyn, N. Y. (Age 21.) Refers to T. Saville, C. T. Schwarze, D. S. Trowbridge.

PENNOTTI, ANGELO RAYMOND, New York City. (Age 22.) Refers to J. J. Costa, A. V. Sheridan.

PERRICELLI, ERNANDO FRANK, New York City. (Age 24.) Refers to F. E. Foss, M. H. Van Buren.

PETITMERMET, PAUL JULES, Newton, Mass. (Age 21.) Refers to C. B. Breed, C. M. Spofford.

PFAFF, LLOYD ALBERT, New Lenox, Ill. (Age 20.) Refers to J. J. Doland. A. J. Hammond, W. C. Huntington.

PHILLIPS, MARION EDGAR, Wichita, Kans. (Age 25.) Refers to L. E. Conrad, F. F. Frazier, M. W. Furr, F. C. Hingsburg, J. M. Phillips, L. V. White.

PHILLIPS, NORMAN, Jr., Evington, Va. (Age 23.) Refers to R. B. H. Begg, F. J. Sette.

PIKE, THOMAS OLIVER, Winters, Tex. (Age 24.) Refers to J. T. L. McNew, J. J. Richey.

PILKINGTON, GEORGE BROWN, Marianna, Ark. (Age 23.) Refers to R. P. Black, H. K. Gatley, F. C. Snow.

PITZEN, TRACY ALLEN, Waterloo, Iowa. (Age 25.) Refers to A. H. Fuller, W. E. Galligan.

POORMAN, MARY ESTHER, West Lafayette, Ind. (Age 21.) Refers to W. K. Hatt, S. C. Hollister, M. S. Ketchum, G. E. Lommel, R. B. Wiley, C. C. Williams.

POPOFF, EGOR PAUL, San Francisco, Cal. (Age 20.) Refers to R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, S. T. Harding, C. G. Hyde.

POTTS, CLIFFORD BERNARD, Yosemite, Cal. (Age 28.) With U. S. Bureau of Public Roads. Refers to H. A. Alderton, Jr., W. T. Collings, Jr., H. J. Gault, D. L. MacBeath, G. L. McLane, W. J. Nelson.

PUGH, CLIFFORD ARTHUR, Long Beach, Cal. (Age 23.) Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder, W. H. Thoman.

PUGH, WALLACE RAYMOND, Long Beach, Cal. (Age 25.) Refers to C. L. Eckel, E. W. Raeder.

RAMBONE, NICHOLAS ARTHUR, New York City. (Age 23.) Refers to J. J. Costa, A. V. Sheridan.

RANTZ, SAUL EDWARD, Philadelphia, Pa. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

RATHVON, SAMUEL FORNEY, Anaheim, Cal. (Age 22.) Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder, W. H. Thoman.

REED, GARNER WADE, Brooklyn, N. Y. (Age 28.) Refers to H. P. Hammond, L. F. Rader, E. J. Squire, J. M. C. van Hulsteyn.

REISNER, ROBERT HENRY, Portland, Ore. (Age 26.) Refers to J. R. Griffith, C. A. Mockmore, H. S. Rogers.

REYES, RAFAEL, Bronx, N. Y. (Age 28.) Refers to R. E. Goodwin, J. C. Rathbun.

RICKENBERG, EDWARD HENRY, Napolean, Ohio. (Age 22.) Refers to G. H. Elbin, A. R. Webb.

ROACH, HAROLD, Los Angeles, Cal. (Age 21.) Refers to R. R. Martel, W. W. Michael, A. L. Sonderegger F. Thomas.

ROBISON, FRANK WILBUR, San Francisco, Cal. (Age 29.) Locating Engr. (truck trail), U. S. Forest Service. Refers to C. Derleth, Jr., C. G. Hyde, C. L. Young.

ROESEL, ERWIN JOSEPH, Kirkwood, Mo. (Age 21.) Refers to H. Cross, J. J. Doland, W. C. Huntington, W. H. Rayner, J. Vawter.

RONAN, EDWARD SANFORD, Jr., Middleburgh, N. Y. (Age 21.) Refers to W. J. Cox, C. S. Farnham, R. S. Kirby, R. H. Suttie, J. C. Tracy.

RONKA, ARNE HENRY, Gloucester, Mass. (Age 33.) Refers to H. K. Barrows, C. B. Breed, A. R. Stuckey, R. G. Taber, H. C. Whitmore.

ROTHERMEL, JOHN BUTLIN, Wyomissing, Pa. (Age 24.) Refers to W. H. Barton, Jr., C. E. Myers.

ROWE, ROBERT RICH, Buffalo, N. Y. (Age 36.) Field Supt. of Constr., Montgomery, Ward & Co., Chicago, Ill. Refers to S. E. Bradford, C. S. Lambie, N. L. Nussbaumer, E. F. Sibbert, C. M. Spofford.

RUSH, BENJAMIN FRANKLIN, Honolulu, Hawaii. (Age 40.) Chf. Engr., Dept. of Public Works, City and County of Honolulu. Refers to W. E. Armstrong, H. A. R. Austin, G. M. Collins, R. L. Garlinghouse, J. F. Kunesh, F. Ohrt, R. Quinn, S. W. Tay, L. J. Watson.

RYDER, LINCOLN WOODBURY, Wollaston, Mass. (Age 21.) Refers to J. B. Babcock, 3d, H. K. Barrows, C. B. Breed, E. Mirabelli, C. M. Spofford.

SACHSE, ALFRED JOHN, De Smet, Idaho. (Age 24.) Refers to I. N. Carter, I. C. Crawford, J. W. Howard, J. A. Murphy.

ST. CLAIR, WILLIAM THADDEUS, Nashville, Tenn. (Age 29.) Refers to R. B. H. Begg, W. R. Glidden, M. Goodkind, E. H. Harder, A. R. Jessup, A. H. Reeves, R. J. Warne.

SALNER, BENJAMIN, Philadelphia, Pa. (Age 21.) Refers to H. L. Bowman, S. J. Leonard.

SAMPLE, MATHIAS WILLIAM, Chuquicamata, Chile. (Age 46.) Mine Supt., Chile Exploration Co. Refers to R. E. Baylor, A. T. Cook, A. Diamant, L. A. Greenley, L. K. Rourke, W. E. Rudolph, T. D. Sawyer, P. A. Seibert, F. Thomas, A. L. Wilcox, S. B. Williamson.

SANDVOS, SIDNEY HALE, Seattle, Wash. (Age 23.) Refers to G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

SAYCHOOK, ROBERT (BORIS) ALLAN, Tiflis, Transcaucasia, U. S. S. R. (Age 25.) Engr., Transcaucasian Scientific and Experimental Inst. of Eng. Constr. (ZIS). Refers to J. A. Beemer, A. P. Davis, C. C. Fisher, C. R. Oldberg, V. V. Tchikoff, L. D. Wilbur.

SCHAEFER, WALTER ARTHUR, Elizabeth, N. J. (Age 23.) Refers to H. E. Breed, E. G. Hooper, C. T. Schwarze.

SCHIFLETT, HUBERT LEE, Denison, Tex. (Age 25.) Refers to E. C. H. Bantel, P. M. Ferguson, J. A. Focht, T. U. Taylor, F. M. Thomson.

SCHOOLCRAFT, GEORGE BINGLY, Clarksdale, Miss. (Age 26.) With L. A. Wilson & Co., Ottawa, Ill. Refers to F. A. Barnes, J. B. Bassett, E. N. Burrows, J. E. Perry, P. H. Underwood.

SCHUCKER, GEORGE WAYNE, Baltimore, Md. (Age 21.) Refers to J. H. Gregory, J. T. Thompson.

SCHWARTZ, PRESTON MacKELLAR, Sierra Madre, Cal. (Age 25.) Refers to R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, S. T. Harding, C. G. Hyde, G. E. Troxell.

SCOTT, JOHN DEAL, Blacksburg, S. C. (Age 21.) Refers to D. D. Curtis, H. E. Glenn.

SCRIVENS, WALTER AMBROSE, Jr., Hackensack, N. J. (Age 21.) Refers to H. N. Cummings, W. S. LaLonde, Jr.

SEARCH, CHARLES OSCAR, Ft. Montgomery, N. Y. (Age 25.) Refers to H. L. Bowman, S. J. Leonard.

SEIFERTH, SOLIS, New Orleans, La. (Age 38.) Weiss, Dreyfous & Seiferth, Inc., Archts. Refers to D. Derickson, W. B. Gregory, B. H. Grehan, P. C. Kuhn, J. A. Miller, G. P. Rice, L. A. Staples.

SETIAN, HAIG, Philadelphia, Pa. (Age 23.) Refers to H. L. Bowman, S. J. Leonard.

SEIBERT, HENRY EDWARD, Brooklyn, N. Y. (Age 22.) Refers to C. S. Farnham, R. H. Suttie, J. C. Tracy.

SIMPSON, ALBERT DEE, Jr., Houston, Tex. (Age 31.) With United Gas System. Refers to J. S. Broyles, R. J. Cummins, V. E. Hamilton, Jr., G. H. Lacy, R. C. S. Watson.

SIMPSON, GEORGE ELLIOTT, Darien, Conn. (Age 26.) Refers to P. S. Dow, R. Fletcher, M. M. Liebeskind, R. R. Marsden, A. P. Richmond, Jr., G. Simpson, J. E. Wadsworth.

SMALLWOOD, DAVID MONTGOMERY, Philadelphia, Pa. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

SMITH, NEWMANN BREEDEN, Redlands, Cal. (Age 30.) Asst. Engr. Metropolitan Water Dist. of Southern California. Refers to J. B. Bond, W. Bowie, J. F. Brookes, E. O. Heaton, F. W. Hough, W. B. James, C. J. Shults.

SMYTH, SID, Phoenix, Ariz. (Age 52.) Deputy State Highway Engr., Arizona State Highway Dept. Refers to F. W. Flittner, R. A. Hoffman, V. H. Housholder, G. L. McLane, T. Maddock, T. S. O'Connell.

SNYDER, ALBERT FREDERICK, Brooklyn, N. Y. (Age 26.) Refers to H. P. Hammond, E. J. Squire.

SOMMER, MATTHEW MICHAEL, Philadelphia, Pa. (Age 24.) Refers to H. L. Bowman, S. J. Leonard.

SPICER, CHARLES BRYSON, Los Angeles, Cal. (Age 26.) Refers to F. J. Converse, R. R. Martel, W. W. Michael, F. Thomas.

SPORSEEN, STANLEY EMANUEL, Seattle, Wash. (Age 29.) Refers to I. L. Collier, G. E. Hawthorn, A. F. Marion, C. C. May, C. C. More, R. G. Tyler.

STITES, HOWARD INGHAM, Burbank, Cal. (Age 32.) City Engr. Refers to J. C. Albers, R. L. Anderson, W. R. Armstrong, H. E. Hedger, H. W. Hincks, A. Jones, H. M. Jones, W. T. Knowlton, C. W. Paige.

STOLIAROFF, NICHOLAS SERGE, Bronx, N. Y. (Age 22.) Refers to H. E. Breed, A. Haring, E. G. Hooper, T. Saville, C. T. Schwarze, D. S. Trowbridge.

STRANIX, JOHN THOMAS, Silverton, Ore. (Age 24.) Refers to J. R. Griffith, H. S. Rogers.

SUENDERMAN, HENRY, Colma, Cal. (Age 23.) Refers to C. G. Hyde, B. Jameyson.

SULKOWSKI, WALTER VALENTINE, Buffalo, N. Y. (Age 25.) Refers to E. F. Berry, L. Mitchell, S. D. Sarason.

SULLIVAN, EUGENE MICHEAL, Centerport, N. Y. (Age 27.) Eng. Inspector, New York State Transit Comm. Refers to N. S. Covert, H. M. Leon, W. L. Selmer, G. C. Whitney, J. F. Williamson.
J. B. Wells, C. R. Young.

SUTHERLAND, JOHN GORDON, Atherton, Cal. (Age 25.) Acting Instructor, Dept. of Civ. Eng., Stanford Univ. Refers to J. C. L. Fish, C. H. Mitchell, L. B. Reynolds,

SUTHERLAND, MARVIN McTYIERE, Pulaski, Va. (Age 21.) Refers to J. L. Newcomb, L. H. Williamson.

SWANSON, CARL GUSTAVE WALTER, New York City. (Age 24.) Refers to J. B. Babcock, 3d, H. K. Barrows, C. M. Spofford.

SWIFT, WILLIAM PATRICK, Bronx, N. Y. (Age 24.) Refers to J. J. Costa, A. V. Sheridan.

SYLVESTER, JACK, Pasadena, Cal. (Age 24.) Refers to R. M. Fox, D. M. Wilson.

TAHSIN, METMET ALI, Urbana, Ill. (Age 25.) Refers to J. S. Crandell, W. C. Huntington, C. E. Palmer.

TATUM, FRED ERWIN, Nora Springs, Iowa. (Age 22.) Refers to A. H. Fuller, F. Kerekew.

TAUL, HORACE WAYNE, Calwa, Cal. (Age 24.) Refers to J. B. Babcock, 3d, C. B. Breed, T. R. Camp, W. M. Fife, C. M. Spofford.

TEAGUE, JAMES EDWARD, Whitwell, Tenn. (Age 23.) Refers to N. W. Dougherty, H. H. Hale.

TENCH, RUDOLPH CLAUDE, Richmond, Va. (Age 25.) Asst. Engr., Chesapeake & Ohio Ry. Co. Refers to A. C. Copland, G. K. Davis, C. W. Johns, A. C. Matthews, Jr., C. Miller, V. B. W. Poulsen, L. L. Shirey.

THAYER, EARL EDWARD, East Lansing, Mich. (Age 27.) Refers to C. L. Allen, L. J. Rothgery.

THOMAS, CYRIL, Woodland, Cal. (Age 30.) Refers to J. C. L. Fish, E. L. Grant, H. H. Hall, L. B. Reynolds, J. B. Wells

TORIBIO, SIMEON GALVEZ, Manila, Philippine Islands. (Age 27.) With Dept. of Public Works and Communications. Refers to R. M. Fox, D. M. Wilson.

TUCKER, WILLIAM BURNS, Richmond, Cal. (Age 20.) Refers to R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, C. G. Hyde, O. W. Peterson.

TYBURSKI, LEO THADDEUS, Wilkes-Barre, Pa. (Age 23.) Refers to H. K. Barrows, E. R. Cary, W. M. Fife, G. G. Jacobosky, E. Mirabelli, W. W. Rousseau, H. O. Sharp, C. M. Spofford, G. B. Walker.

UHRICH, MORRIS BORDNER, Montpelier, Vt. (Age 21.) Surveyor, U. S. Engr. Dept. Refers to M. O. Fuller, H. G. Payrow, C. H. Sutherland.

VAN LIEW, ROSCOE EMERSON, Salt Lake City, Utah. (Age 24.) Refers to F. M. Allen, G. M. Bacon, E. K. Brown, M. Housecroft, R. B. Ketchum, F. H. Richardson.

VLADIMIR, CLARENCE, Seattle, Wash. (Age 28.) Refers to C. W. Harris, C. C. More, R. G. Tyler.

WAHL, GEORGE RENE ROBERT, East Keansburg, N. J. (Age 22.) Refers to G. M. Fair, A. Haertlein, L. J. Johnson.

WALKER, JACOB J, Jr., Bowling Green, Ohio. (Age 29.) Chf. Deputy Surveyor-County Surveyor, Wood County; also City Engr., Bowling Green, Ohio. Refers to H. P. Jones, N. W. Locke, S. C. McKee, J. B. Morrison, G. N. Schoonmaker, R. E. Shultz, H. M. Sharp, H. A. Stepleton, R. M. Strohl.

WALTON, EDWARD HAVILAND, Swarthmore, Pa. (Age 20.) Refers to E. M. Bassett, S. B. Lilly.

WATTS, CLEAL THOMAS, Des Moines, Iowa. (Age 34.) Refers to R. Budd, C. H. Currie, J. S. Dodds, A. Epstein, J. E. Van Liew.

WEINBERG, LOUIS DAVID, Lansing, Ill. (Age 21.) Refers to H. E. Babbitt, E. E. Bauer, J. J. Doland, W. C. Huntington, F. W. Stubbs, Jr.

WELLNER, JOHN ARNOLD, Oak Park, Ill. (Age 25.) Refers to J. J. Doland, W. C. Huntington, A. Jorgensen, F. W. Stubbs, Jr.

WERNER, NILES, Boulder City, Nev. (Age 43.) Designer, Six Companies, Inc. Refers to A. H. Ayers, S. B. Barnes, A. Donaldson, R. P. Miller, A. G. Roach, R. H. Taylor.

WESTAWAY, CLARENCE ROYAL, Canton, N Y. (Age 24.) Refers to C. B. Breed, C. M. Spofford.

WHITNEY, LAWRENCE WOODWARD, Brooklyn, N. Y. (Age 23.) Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

WIESNER, HENRY, Jr., Pleasanton, W. Va. (Age 28.) Refers to G. P. Boomsliter, L. V. Carpenter, H. O. Cole, R. P. Davis, W. S. Downs.

WILKENS, WERNER, Jr., San Antonio, Tex. (Age 32.) With Armco International Corporation, Sao Paulo, Brazil. Refers to J. C. Carpenter, C. L. Eckel, G. M. de Menezes, S. B. Moore, J. B. Morrison, M. C. Patton, E. W. Raeder, G. E. Shafer, C. Q. Simoes.

WILLIS, LOVELL, New Haven, Conn. (Age 22.) Refers to C. T. Bishop, J. C. Tracy.

WILSON, CARL AMOS, Kansas City, Kans. (Age 31.) Asst. to County Engr., Wyandotte County, Kans. Refers to G. W. Bradshaw, J. O. Jones, W. C. McNown.

WINCHESTER, ERIC WILLIAM, Philadelphia, Pa. (Age 22.) Refers to H. L. Bowman, S. J. Leonard.

WINSCHUH, ALBERT JOSEPH, Totowa Borough, N. J. (Age 28.) Refers to E. R. Cary, L. W. Clark, T. R. Lawson, W. W. Rousseau, H. O. Sharp.

WINSMORE, KARL BUHRE, Ridley Park, Pa. (Age 22.) Refers to L. W. Clark, T. R. Lawson, H. O. Sharp.

WISSMILLER, IVAN LEE, Cookesville, Ill. (Age 23.) Refers to H. Cross, J. J. Doland, W. C. Huntington, H. H. Jordan, G. W. Pickels, F. W. Stubbs, Jr.

WITMER, EARL JOHN, Lancaster, Pa. (Age 25.) Refers to H. L. Bowman, S. J. Leonard.

WONG, YUE-KWONG, Pasadena, Cal. (Age 23.) Refers to R. R. Martel, W. W. Michael.

WOODRUFF, JOHN DEWEY, Dodge City, Kans. (Age 23.) Refers to L. E. Conrad, F. F. Frazier, M. W. Furr, W. A. Keene, Jr., L. V. White.

YARGER, WILLIAM LEE, Bloomington, Ill. (Age 26.) Refers to J. S. Crandell, J. J. Doland, W. C. Huntington, W. H. Rayner, F. W. Stubbs, Jr.

YOUNG, JOE WAH, Shanghai, China. (Age 32.) Engr. with Palmer & Turner, Archts., Surveyors & Civ. Engrs. Refers to C. B. Breed, R. V. Giles, K-S. Hsu, C-P Hsueh, R. T. Lassiter, J. L. Piland, C. M. Spofford.

ZANDER, PAUL MARTIN, Champaign, Ill. (Age 23.) Refers to H. Cross, W. A. Oliver.

ZIEGLER, ROBERT NEWTON, Cincinnati, Ohio. (Age 23.) Rodman, Hamilton County (Ohio) San. Eng. Dept. Refers to R. A. Anderegg, H. H. Kranz, H. B. Luther, C. N. Miller, J. S. Rafferty.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

ADAMS, WILLIAM NEIL, Assoc. M., Ames, Iowa. (Elected May 28, 1923.) (Age 47.) Chf. Bridge Designer, Iowa State Highway Comm. Refers to R. A. Caughey, C. C. Coykendall, F. H. Mann, A. Marston, F. R. White.

CRADDOCK, FRANKLIN HARPER, Assoc. M., Olympia, Wash. (Elected Junior Sept. 10, 1918; Assoc. M. Oct. 10, 1921.) (Age 43.) Office Engr., Washington Dept. of Public Works. Refers to C. J. Bartholet, M. E. Clark, J. W. Cunningham, H. L. Gray, C. D. Pollock, G. W. Rupp, R. K. Tiffany.

CRITES, GEORGE SOLOMON, Assoc. M., Punxsutawney, Pa. (Elected Aug. 31, 1915.) (Age 52.) Div Engr., Baltimore & Ohio R. R. Co. Refers to T. Bloecher, Jr., H. T. Cory, W. R. Edwards, H. H. Harsh, P. G. Lang, Jr., E. Stimson.

DUNHAM, JOHN WETHERBEE, Assoc. M., Washington, D. C. (Elected Junior Jan. 15, 1923; Assoc. M. June 10, 1929.) (Age 35.) Structural Engr., Office of Superv. Archt., Treasury Dept. Refers to W. S. Anderson, L. N. Edwards, F. C. Hilder, A. W. Santelmann, P. D. Sargent, G. G. Sloane.

EREMIN, ANATOL ALEXANDER, Assoc. M., Sacramento, Cal. (Elected March 15, 1926.) (Age 39.) Associate Bridge Designing Engr., California State Highway Comm. Refers to C. E. Andrew, N. D. Baker, M. C. Collins, R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, F. W. Panhorst.

FLITTNER, FRANK WILLIAM, Assoc. M., Phoenix, Ariz. (Elected Junior Nov. 21, 1921; Assoc. M. March 12, 1923.) (Age 40.) Associate Constr. Engr., U. S. Bureau of Public Roads. Refers to J. De Arozena, W. R. Hutchins, E. V. Miller, T. S. O'Connell, J. W. Powers, J. R. Van Horn.

FOIGHT, CLARENCE DOUGLAS, Assoc. M., Pittsburgh, Pa. (Elected Aug. 30, 1926.) (Age 38.) Designing Engr., Div. of Bridges, Bureau of Eng. Refers to H. S. Ayres, J. T. Campbell, D. E. Davis, D. B. Steinman, J. D. Stevenson, H. A. Thomas, C. M. Vetter.

GOODMAN, WALLACE SHUFELDT, Assoc. M., San Antonio, Tex. (Elected Nov. 10, 1930.) (Age 46.) County Engr., Bexar County, Tex. Refers to A. R. Alling, E. P. Arneson, C. T. Bartlett, H. R. F. Helland, A. J. McKenzie, F. E. Rightor, E. W. Robinson.

KETTER, E F, Assoc. M., Gary, W. Va.. (Elected Aug. 28, 1922.) (Age 47.) Chf. Draftsman, U. S. Coal & Coke Co. Refers to S. C. Baker, C. W. Brown, H. N. Eavenson, C. V. R. Fullenwider, J. R. Hicks, A. L. Hyde.

KIEHNLE, WILLIAM ALBERT, Assoc. M., New York City. (Elected Junior March 12, 1923; Assoc. M. Aug. 27, 1928.) (Age 35.) Secy. and member of firm, William V. Kiehnle Co., Bldrs. and Engrs. Refers to A. Fellheimer, T. Heatley, K. Hoyt, C. T. Schwarze, C. H. Snow.

LAWLOR, JOSEPH PATRICK, Assoc. M., Ames, Iowa. (Elected Aug. 4, 1924.) (Age 39.) Vice-Pres. and Gen. Mgr., Layne North Central Co., Forest Park, Ill. Refers to J. H. Ames, R. S. Charles, A. S. Fry, P. C. Gauger, J. G. Gordon, Jr., R. E. Hutchins, C. R. Knowles, R. E. McDonnell, E. E. Michaels, F. V. Ragsdale, H. G. Suehrstedt. H. A. Wiersema.

McKEAN, HARRY PARKER, Assoc. M., Chicago, Ill. (Elected Junior June 6, 1911; Assoc. M. Nov. 26, 1918.) (Age 46.) Civ. Engr., Harza Eng. Co. Refers to J. S. Bowman, F. A. Dale, C. P. Dunn, G. W. Hamilton, L. F. Harza, T. Pettersen, H. A. Rands.

MOFFETT, JOHN WILLIAM, Assoc. M., Philadelphia, Pa. (Elected Sept. 11, 1917.) (Age 45.) Chf. Structural Engr., United Engrs. & Constrs., Inc. Refers to J. D. Anderson, F. O. Dufour, J. W. May, G. Schobinger, S. H. Widdicombe.

MORRIS, EARLE HEDDERICH, Assoc. M., Charleston, W. Va. (Elected April 3, 1922.) (Age 46.) Chf. Engr., Public Service Comm. of West Virginia. Refers to E. F. Chandler, H. O. Garman, E. R. Griffin, C. Johnson, C. M. Ross, J. E. Settle.

PITTMAN, EVERETTE EDWARD, Assoc. M., Austin, Tex. (Elected Jan. 14, 1929.) (Age 41.) Res. Engr., State Highway Dept. of Texas. Refers to E. C. H. Bantel, G. G. Edwards, J. A. Focht, G. Gilchrist, O. A. Seward, Jr., M. C. Welborn, G. G. Wickline.

REED, PERCY LAWRENCE, Assoc. M., Gainesville, Fla. (Elected June 11, 1917.) (Age 58.) Prof. and Head of Dept., Coll. of Eng., Univ. of Florida. Refers to R. M. Angas, C. C. Brown, F. O. Dufour, R. Farnham, W. W. Fineren, H. D. Mendenhall, W. L. Wilson.

STEARNS, JOHN, Assoc. M., Banning, Cal. (Elected Aug. 31, 1915.) (Age 51.) Div. Engr., Div. No. 3, Metropolitan Water Dist. of Southern California. Refers to E. A. Bayley, J. Hinds, W. W. Hurlbut, S. R. Jones, S. B. Morris, H. A. Van Norman, E. G. Williams.

TAYLOR, NELSON, Assoc. M., Los Angeles, Cal. (Elected Junior Sept. 5, 1911; Assoc. M. July 2, 1913.) (Age 47.) Member of firm, Salisbury, Bradshaw & Taylor, Cons. Engrs. Refers to W. K. Barnard, C. Bradshaw, R. A. Hill, A. L. Sonderegger, F. E. Trask, J. T. Vawter.

FROM THE GRADE OF JUNIOR

BOOTH, ARCHIBALD ALLAN KIRSCHNER, Jun., Newfield, N. Y. (Elected July 11, 1927.) (Age 31.) Landscape Foreman, Enfield Park, N. Y. Refers to F. L. Castleman, J. P. Churchill, E. A. Dockstader, F. E. Hertel, G. E. Howe, F. C. Moore, H. W. Preston, I. W. Pugh, H. T. Rights, L. J. Towne, H. A. Vanderbeek.

BROADFOOT, HOLLIS LEE, Jun., Florence, Ala. (Elected June 7, 1926.) (Age 32.) Refers to H. F. Anthony, H. D. Burnum, T. A. Forbes, W. M. Hall, W. N. Nelson, E. A. Rudulph, E. J. Trimble.

CLELAND, RALPH RANKIN, Jun., State College, Pa. (Elected Oct. 14, 1929.) (Age 28.) San. Engr. with Pennsylvania State Coll. Refers to H. K. Kistler, R. O'Donnell, W. P. Rothrock, H. B. Shattuck. E. D. Walker.

FARKAS, NICHOLAS, Jun., Bronx, N. Y. (Elected April 23, 1928.) (Age 32.) Refers to C. E. Conover, J. H. Quimby, A. I. Raisman, P. L. Voss, J. P. J. Williams.

HAVELIN, JULIUS EDWARD, Jun., Philadelphia, Pa. (Elected July 9, 1923.) (Age 32.) Engr., Philadelphia Elec. Co., Mech. Eng. Div. Refers to H. R. Buck, F. W. Deck, T. B. Rights, F. J. Seery, G. E. Sissons, C. M. Wetzel.

KELKER, JAMES JOSEPH ARTHUR, Jun., Shreveport, La. (Elected Jan. 13, 1930.) (Age 27.) Civ. Engr., Mid-Kansas Oil & Gas Co. Refers to E. L. Erickson, W. K. Hatt, W. J. Henderson, G. E. Lommel, R. B. Wiley.

MALE, MILTON, Jun., New York City. (Elected Oct. 14, 1929.) (Age 28.) Tech. Asst., U. S. Steel Corporation. Refers to O. E. Hovey, F. T. Llewellyn, L. S. Moisseiff, H. M. Priest, S. Weishoff, W. H. Weiskopf.

MONROE, CLARENCE ALBERT, Manhasset, N. Y. (Elected Jan. 13, 1930.) (Age 32.) Secy. and Treas., Manhasset Civ. Engrs., Inc. Refers to A. G. Bancker, F. J. Biele, W. N. Brown, T. J. Carlile, J. L. Herber, T. E. Ringwood.

MORGAN, BENJAMIN ARTHUR, Jr., Jun., Balboa, Canal Zone. (Elected July 25, 1932.) (Age 29.) Concrete Inspector, Madden Dam Div., The Panama Canal. Refers to J. E. Deignan, B. J. Fletcher, A. L. Hertz, R. L. Klotz, L. B. Moore, R. F. Olds, E. S. Randolph, M. A. Seiler.

OLEWILER, WILLIAM NORMAN, Jun., Ossining, N. Y. (Elected Dec. 15, 1924.) (Age 32.) Asst. Engr., Applebee & Slater, Engrs. Refers to H. W. Bressler, J. F. Sanborn, W. S. Smith, Jr., E. D. Walker, C. R. Wyckoff.

PONCO, JACINTO SAMONTE, Jun., Manila Philippine Islands. (Elected Oct. 14, 1930.) (Age 32.) Asst. Civ. Engr., Div. of Architecture, Bureau of Public Works. Refers to W. K. Hatt, L. C. Urquhart, R. B. Wiley. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

SAMSON, WALTER HOWARD, Jun., Honolulu, Hawaii. (Elected Oct. 14, 1929.) (Age 32.) Engr., Board of Water Supply. Refers to W. E. Armstrong, J. F. Kunesh, G. K. Larrison, F. Ohrt, C. D. Tarleton, A. C. Wilson.

SANTOS, TIRSO NOLLORA, Jr., Jun., Washington, D. C. (Elected Oct. 1, 1928.) (Age 31.) Asst. Structural Engr., Office of Superv. Archt., Treasury Dept. Refers to J. W. Dunham, J. H. Eaton, C. Mayer, C. M. Spofford, C. H. Sutherland.

WEED, ELLSWORTH SAMUEL, Jun., New Cumberland, Pa. (Elected Oct. 1, 1926.) (Age 32.) Asst. Engr., Water and Power Resources Board, Harrisburg, Pa. Refers to G. S. Beal, H. E. Beckwith, N. S. Buckbee, H. V. Gulick, T. E. Seelye, F. H. Weed.

WRIGHT, WILLIAM THEODORE, Jun., Santa Monica, Cal. (Elected Dec. 3, 1928.) (Age 28.) Structural Engr., T. C. Kistner & Co., Archts., Los Angeles, Cal. Refers to H. G. Balcom, J. L. Edwards, H. M. Hadley, L. R. Hjorth, C. C. More, G. A. Sallans, W. F. Way.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.